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AND

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

EXPERIMENTAL OBSERVATIONS ON GROUTING SANDS AND GRAVELS

BY ALFRED MACHIS,[1] JUN. ASCE

SYNOPSIS

The laboratory research reported in this paper was conducted to provide data on the use of cement slurries to prevent the contamination of water wells. The initial impetus for this investigation was the acute situation resulting from the infiltration of salt water into wells that threatened, severely, the quality of the Baltimore, Md., underground waters.

A water-bearing formation may be contaminated by salt water in one of two ways. If the formation has an outcrop under the sea, the sea may move into the formation when excessive pumping reduces the internal pressure. Such a situation may exist in some parts of the upper aquifers of the Baltimore industrial area. However, there is no way in which this condition may be alleviated by the use of cement. Many industries which originally had wells in these formations abandoned them and drove new wells tapping deeper fresh-water strata.

In many instances the new wells also became contaminated by salt water and the question then arose as to how the salt water entered the new formations. Investigation revealed that in general the deeper formations were not receiving salt water directly from the sea but were being contaminated chiefly by salt water leaking down between the well casing and the drilled hole from the upper strata. This leakage may at first seem impossible since almost all wells are drilled through clay beds. However, it has been shown by the use of caliper logs that the actual diameter of a drilled well usually is considerably larger than the diameter of the bit. Therefore, the reason why leakage does occur down the outside of a casing set into such a hole should be obvious. In some instances this leakage occurs in the same well that is producing the water; in other cases, it may occur in some distant well, often an abandoned one, and

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by March 1, 1947.

[1] Formerly 1st. Lt. Corps of Engrs., U. S. Army, Baltimore, Md.

may then pass into the producing formation. For cases of leakage around the outside of the well casing, grouting may be used to prevent further deterioration of water quality.

The seriousness of the situation was emphasized in 1944 by John C. Geyer,[2] Assoc. M. ASCE. In some instances the wells were producing water with a salt content as high as 3,000 ppm whereas other wells in the same formation contained only from 8 ppm to 12 ppm of salt.

The problem cannot be solved by abandoning (without sealing) the contaminated wells and drilling new ones. The capital investment in the average well of this region is about $15,000. On the other hand, a well may be repaired successfully in most cases for less than 10% of $15,000. In this light it becomes difficult to justify the extra expenditure of drilling a new well; and, in addition, an abandoned well that has not been properly sealed is a greater hazard to the area as a whole than is the same well when it is being pumped. In the latter case the well may serve to contaminate all the lower fresh-water formations by acting as an open conduit between the strata. In fact such cases as this do exist and are increasing materially the difficulty of solving the salt-water contamination problem.

Grouting New Casing

Many methods may be used to grout new casing into a well. Essentially, however, all these methods fall into one of the three main classes known as bailer, tubing, or casing methods. In each method the cement mixture commonly employed is a "neat" proportion of water and cement without any addition of sand. This mixture is also variously referred to as "cement slurry" or "cement grout." In all cases, the cement is mixed with the desired proportion of water before it is allowed to pass into the well.

Dump Bailer Method of Cementing.—In the dump bailer method, the slurry is lowered to the bottom of the well in a cylindrical container and is emptied through a foot valve. This method of cementing is too limited in its scope and, therefore, has been superseded by improved methods.

Tubing Method of Cementing.—In the tubing method of cementing, the slurry is pumped into the bottom of the well through a tube down the center of the well. Since the cement must be forced out around the outside of the casing, it is obvious that there must be a free passage for the entire length of the casing. This condition is usually insured by establishing the free circulation of water down around the casing shoe and up to the surface.[3]

Casing Method of Cementing.—In this method of cementing, the slurry is pumped directly through the casing down to the bottom of the well. The

[2] "Ground Water in the Baltimore Industrial Area," by J. C. Geyer, Maryland State Planning Comm., Baltimore, May, 1945.

[3] "Methods of Shutting Off Water in Oil and Gas Wells," by F. B. Tough, *Bulletin No. 163*, Bureau of Mines, U. S. Dept. of Commerce, Washington, D. C., 1918, plate facing p. 38.

slurry is usually enclosed between two moving plugs to keep it from becoming diluted or contaminated.[4]

Repair of Leaking Wells

The repair of a leaking well is a much more difficult problem than the grouting of a new casing. Where possible, the casing of a leaking well should be pulled and the well should be grouted in the same manner as a new well. This is seldom possible, however, because the casing will usually be tightly bound by cavings of the earth wall. For this same reason, it may also be impossible to establish a circulation of water or mud around the outside of the casing. For such a circumstance the method of "squeeze cementing" developed in the oil industry must be applied. "Squeeze cementing" is the name applied to a process whereby the cement slurry is forced into, or against, a formation through perforations in the casing. The method is very similar to the tubing method of cementing except that the slurry is forced directly against the salt-water formation instead of down around the casing shoe. The highly successful utilization of this process in the oil well industry warrants close study for possible adaptation to the water well industry.

Sealing Abandoned Wells

The sealing of wells which are to be abandoned is highly essential to prevent them from acting as open conductors between fresh and contaminated water-bearing formations. It is first necessary to pull all casing that is not too securely bound in the formation. The remaining sections must then be completely slotted or perforated so that the slurry can enter behind them at many points, thus insuring the complete filling of all cavities and channels behind the casing. The previous practice of simply filling the inside of the casing has shown itself to be insufficient, for leakage may still follow a path down the outside.

After the casing has been removed or perforated, a process similar to squeeze cementing may be used to seal the well. The hole should be grouted by progressive stages from bottom to top. An attempt made to seal the entire well in one operation without raising the outlet of the grout tube may cause the grout pipe to stick and thus make a failure of the job.

Cement Penetration Through Sands

In determining what happens to the grout that is pumped into a well, the first question that must be answered pertains to the penetration of the cement grout through the sand strata. Thus, with a given sand formation, may the grout be expected to remain in the hole or will it pass into the pores of the sand and away from the well?

In addition to solving the problem of the water well industry, such data should also be valuable in shedding light on the problems of shutting off water leakage through earth dams, levees, railroad embankments, foundations, etc. In the water well industry the problem is to prevent the excessive loss of slurry into the formation; but, in these other cases, the problem is to find the best

[4] "Methods of Shutting off Water in Oil and Gas Wells," by F. B. Tough, *Bulletin No. 163*, Bureau of Mines, U. S. Dept, of Commerce, Washington, D. C., 1918, plate facing p. 48.

method of forcing the grout into the pores of the sand so as to form an impermeable wall.

Materials.—The type of cement used may be a factor in determining the quantity of slurry that will pass into the formation. There are three general grades from which a cement may be chosen—high early strength, regular Portland, or special oil well cement. For this work, a Portland cement was chosen since it is the type most often used.

The characteristics of the sand used will also have an effect upon the penetration of the slurry into the sand. It is easy to understand that a sand with large pores will readily admit the passage of cement slurry whereas a sand with pores smaller than the cement grains will not pass any of the slurry. By considering the pore size and shape together as a single quality of the sand, a measure of this quality may be obtained in the expression of the sand permeability. To secure a wide range of permeabilities, sands of uniform size were used in this work. The permeabilities of all the samples are given in a subsequent section of this paper. Since an expression of the permeability completely defines the hydraulic characteristics of the sand, natural formations with the same permeabilities should react to cement penetration in the same manner as the laboratory samples.

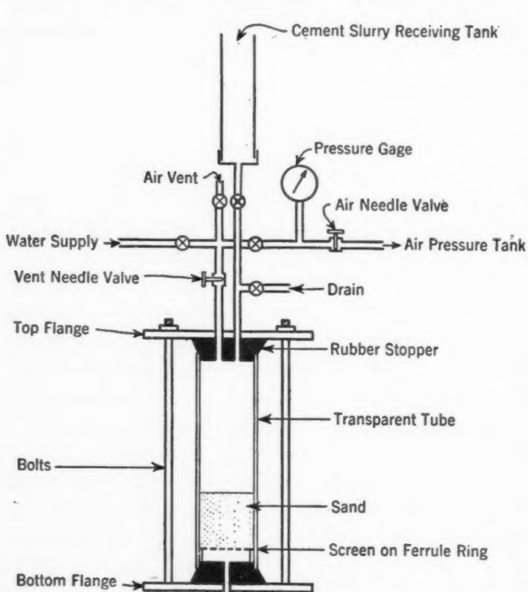

Fig. 1.—Schematic Diagram of Cement Slurry Penetration Apparatus

Other Factors Affecting Penetration.—In addition to the type of sand and cement used, the factors of temperature and pressure may change the properties of the cement slurry and, therefore, also may affect the penetration of the slurry into the sand. In work done by R. Floyd Farris,[5] a number of cements were tested at various temperatures and pressures to determine the effect of these factors on the change of consistency with time. From an examination of the study it appears that, in cementing water wells, the temperature and pressure effects probably will not be of any significance.

[5] "Effects of Temperature and Pressure on Rheological Properties of Cement Slurries," by R. F. Farris, *Transactions*, A.I.M.M.E., Vol. 142, 1941, p. 117.

Apparatus.—The apparatus, shown in Fig. 1, used in this investigation, consisted of a transparent tube of glass or plastic which contained the sand sample being tested. The tubes used were $1\frac{1}{2}$ in. in diameter and from 7 in. to 11 in. long. The sand was supported within the tube by a wire cloth soldered to a ferrule ring. Rubber stoppers, used as gaskets in the ends of the tubing, were squeezed tightly between two steel flanges by four long bolts. The flanges were drilled and tapped at the three points shown, to receive $\frac{1}{4}$-in. steel pipe.

Procedure.—The sand sample to be tested was weighed and poured into the transparent tube. The best procedure for removing all air completely from the sand was to immerse the tube in a beaker of water and to allow the water to rise slowly from the bottom of the sand. The transparent tube was placed in the apparatus and the excess water was allowed to flow out until the meniscus was just at the top of the sand. The sand was vibrated to compact it by tapping the bottom flange with a rubber hammer.

The porosity of each sample was determined by measuring the height of the sample and computing the porosity from the volume and specific gravity of the sand. The cement and water were proportioned to give the desired consistency and mixed. The slurry was poured into the receiving tank from which it could be added to the transparent tube. A fresh slurry was used for each run.

TABLE 1.—Penetration of Cement Slurries through Sands

Slurry[a] (gal per sack) (1)	Porosity n (2)	Air pressure[b] (3)	Dry cement[c] (%) (4)	Volume reduction[d] (5)
(a) Tyler Sieve Nos. 4 to 8 (4.699 Mm to 2.362 Mm)				
6	42.8	30	...	...
6	40.9	30	19.7	0.41
7	40.1	15	85.6	0.03
8	39.4	5	>76[e]	0.13
(b) Tyler Sieve Nos. 8 to 10 (2.362 Mm to 1.651 Mm)				
6	41.0	30	...	...
6	42.3	30[f]	24.8	0.42
7	39.2	20	82.8	0.08
8	39.3	30[f]	31.3	0.44
8	39 2	15	95.4	0.06
8	39.8	5	...	...
9	39.6	30[f]	39.6	0.37
9	39.2	15	92.2	0.06
10	39.8	15	95.0	0.00
10	39.2	25[f]	17.8	0.52
(c) Tyler Sieve Nos. 10 to 14 (1.651 Mm to 1.168 Mm)				
6	39.6	30	...	...
7	40.4	30	...	...
7	39.6	30	26.5	0.38
8	41.3	30	...	...
8	41.5	30	29.0	0.38
9	42.2	20	88.8	0.04
10	41.5	20	86.4	0.04
(d) Tyler Sieve Nos. 14 to 16 (1.168 Mm to 0.991 Mm)				
8	40.8	30	...	...
8	40.7	30	0.6	0.68
8	40.9	30	1.5	0.75
9	41.6	30	...	...
9	41.0	20	79.8	0.06
10	41.0	15	...	...
12.5	41.4	25	...	...
14.5	39.2	30	...	...
14.5	39.9	30	100.	...
(e) Tyler Sieve Nos. 16 to 20 (0.991 Mm to 0.833 Mm)				
7	44.6	30	...	...
8	43.6	30	...	...
8	40 9	30	6.8	0.65
8	40.1	25	2.8	0.68
9	42.2	30	...	...
9	41.2	25	1.2	0.63[g]
10	40.9	30	46.6	0.27
12.5	44.7	30	2.5	...
12.5	41.2	30	79.0	0.07
12.5	40.6	30	...	...
13.5	41.4	30	55.3	0.15
14.5	40.0	10 to 30	...	...
14.5	41.8	20	77.3	0.04
(f) Tyler Sieve Nos. 20 to 28 (0.833 Mm to 0.589 Mm)				
14.5	41.4	30	...	...
14.5	40.8	30	4.1	0.39
20	41.8	20	47.4	0.15
(g) Tyler Sieve Nos. 28 to 30 (0.589 Mm to 0.503 Mm)				
14.5	42.8	30	0	...
(h) Tyler Sieve Nos. 30 to 40 (0.503 Mm to 0.381 Mm)				
14.5	40.9	30	0	...
14.5	39.9	20	0	0.67
14.5	41.2	30	0	...

[a] Gallons of water per sack of cement; in the text referred to as "6-gal slurry," etc. [b] Pounds per square inch. [c] Percentage of dry cement (by weight) that passed the testing apparatus. [d] Ratio of final volume to initial volume of slurry. [e] Test was stopped before the slurry passed through. [f] Pressure was applied slowly. [g] Apparatus clogged; test incomplete.

After the slurry had been placed, the desired pressure was admitted from an air pressure tank, and the outlet valve was opened. The slurry that passed through the sand was caught and dewatered by boiling. In this manner the weight of dry cement that passed could be determined. The initial weight of dry cement added to the sand was computed from the volume and specific weight of the slurry.

To study the effect of the foregoing method of applying the pressure suddenly, some data were also taken, using a slow application of pressure—first by opening the outlet valve and then by slowly applying the pressure. A distinct difference in results was obtained.

Data and Discussion of Results.—The data on the penetration of a cement slurry through a sand are presented in Table 1. The sand size is given as the mesh of the screen through which the sand will pass and the mesh upon which it will be retained. Mesh sizes, in millimeters, for the standard Tyler sieves are given in parenthesis in Table 1.

Effect of Squeeze Pressure.—When a sand sample of a grain size smaller than 28 mesh (0.589 mm) is used, no amount of pressure will make a slurry pass through the sand. Slurries as dilute as 14.5 gal of water per sack of cement (hereinafter called "14.5-gal slurry," etc) were tested, and the results showed that the cement particles would penetrate into the sand only about three or four grain diameters before they lodged in a restricted pore. The pressure then acted only to squeeze the water from the slurry so that in a few minutes it was possible to reduce a 14.5-gal slurry to a 4-gal or 5-gal slurry. A pressure of 1,000 lb per sq in. was used to test this principle and was found to give exactly the same results as a pressure of 10 lb per sq in. The only difference was that the higher pressures squeezed the water from the slurry more rapidly.

When grain sizes greater than 28 mesh are used, the pressure becomes a more important factor. In this case some slurry can be expected to pass through the sand, and the amount passed will depend on how the pressure is applied. With a given slurry and a given sand, a large quantity of slurry may be made to pass through the apparatus by applying a high pressure very rapidly. This procedure was followed in all the tests in which the percentage of dry cement passed was measured, except the four tests given in lines 2, 4, 7, and 10 of Table 1(*b*) (see footnote[f]). In the first three cases mentioned, a pressure up to 30 lb per sq in. was applied at a rate of about 3 lb per sec. In each case the cement particles ceased to pass through before the full 30 lb per sq in. was reached and any additional pressure simply caused the water in the slurry to filter out more rapidly. In the last case about half of this rate of pressure application was used, and a smaller percentage of slurry was passed than was in the other three tests. This condition held in spite of the fact the slurry was more dilute—a factor more conducive to the passage of cement particles.

A comparison of these four tests with the other tests on the same sand where the full pressure application was instantaneous (less than 1 sec) shows that in the latter case the percentage of cement passed is considerably greater. For example, a comparison of lines 4 and 5, Table 1(*b*), each on an 8-gal slurry, shows that a rapid pressure application will cause the weight of dry cement passed to increase from 31% to 95%.

The main reason why this difference in penetration occurs with increased rate of pressure application is probably that, when a cement slurry is allowed to stand quiescent for a few moments, it will thicken. If violently stirred, the slurry will return to its initial thin concentration. It is very probable that the sudden application of pressure causes a violent surge of the lower layer of slurry which destroys the gelling properties. With the fluidity thus increased, the slurry does not bridge across the sand grains and easily passes into the sand.

Effect of Porosity.—A change in porosity of any given sand will serve to change the size of the pore in the sand and, of course, the amount of slurry which will pass through; but a better method of measuring the effect of a change in pore size can be obtained by changing the sand size. For this reason, an attempt was made to compact each sand into as stable a condition as possible and to maintain the same porosity in each test.

Effect of Core Length.—An examination of Col. 5, Table 1 (which gives the ratio between the final volume of the slurry after all the water has been squeezed from it and the initial volume), will show that the cement may be made to filter even on the coarse (4-mesh to 8-mesh) sand. The plug left on the sand is not the result of an inability of the cement particles to move through the pores of the sand, for observation through the glass sample tube shows that the particles in the pores are continually being washed out by the water filtering from the plug above. Instead, this cement plug is caused by an inability of the cement particles to enter the pores as a result of a strong tendency of the cement to bridge across the sand grains. This bridging has sufficient strength to support the cement particles so that the pressure on the slurry simply forces the water out and leaves a solid cement core. Since this filtering depends only on the action at the sand-cement interface, the length of the core has no effect in the results of the penetration tests.

By comparing the results of Col. 5, Table 1, the amount of slurry that will pass into the sand may be appraised. However, in comparing the results of tests using different consistencies of slurries, allowance must be made for shrinkage that would take place because of the loss of the excess water. To facilitate this comparison, tests on shrinkage of the cement slurry are reported in a following section. In addition to this information, the weight of dry cement that passed through the apparatus, computed as a percentage of the cement in the initial slurry volume, is also given in Col. 4, Table 1. These values would be affected to some degree by the void volume of the sample. With the more dilute slurries, the proportion retained in the pores after the sand was blown out with compressed air would be negligible. With the thick slurries, more would be retained in the pores of the sand; but, even then, the values of Col. 4 would be affected only by a slight percentage.

Effect of Sand Size and Slurry Concentration.—The sand size and the slurry concentration are two factors which have a very marked influence on the percentage of slurry passing into the sand. The large sand sizes give large pore sizes into which the grout may pass readily. When the cement is mixed with a large amount of water to give a thin slurry, the cement particles are too dispersed to bridge across the pore and so are carried into the sand. This

bridging of cement particles is increased if the slurry is allowed to remain quiescent above the sand for several minutes.

To aid in giving a better description of the appearance of a grouted sand formation, photographs of a number of the samples were taken. In each instance the squeezing was stopped just before the air blew the cement out of the pores of the sand. The reason for this was to show the appearance of the sand as it would look if it were in a well that had been grouted by one of the methods previously described. The samples shown were removed from the tube and allowed to set. Part of the core was then cut away to show the interior of the sand and slurry. The bar above the letter $\overline{C}$ in Fig. 2 is used to indicate the sand-cement interface so that the part of the core below the letter $\overline{C}$ is the grout used to fill the section between the casing and the sand formation. The core section above the letter $\overline{C}$ is the sand formation penetrated by the grout. Each division on the scale beside the core is 1 mm so that the size of the core is apparent.

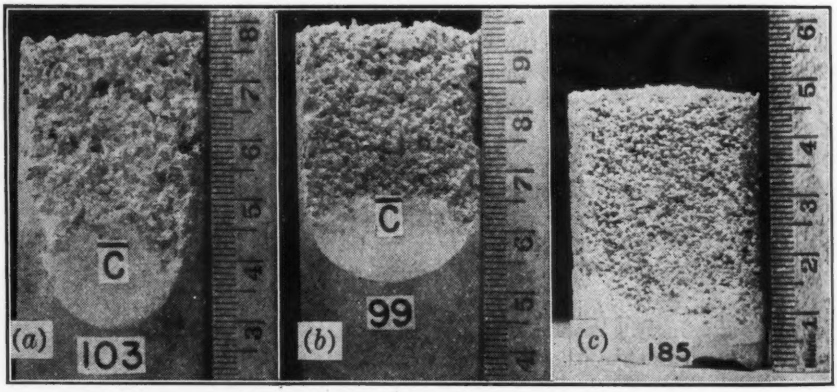

FIG. 2.—RELATIVE TEXTURE OF CORE SPECIMENS GROUTED BY SLURRY

(*a*) No. 8 to No. 10 Sand and 6-Gal Slurry
(*b*) No. 14 to No. 16 Sand and 8-Gal Slurry
(*c*) No. 20 to No. 28 Sand and 20-Gal Slurry

Fig. 2(*a*) shows an 8-mesh to 10-mesh sand grounted with a 6-gal slurry. The slurry has penetrated all the pores, forming a strong solid concrete. The slurry column that has not penetrated the sand has had all the excess water squeezed from it so that it is in its densest and most compact condition. The slurry columns that do not penetrate the sand can always be made to compact into this dense condition regardless of the initial water content.

Fig. 2(*b*) shows a 14-mesh to 16-mesh sand core grouted by an 8-gal slurry. The pores of the sand have become too small to allow the slurry to pass a sufficient percentage of cement to fill the pores completely. In contrast to this case, a 9-gal slurry will pass about 80% of the initial dry weight of cement, and a 14.5-gal slurry will pass 100% of the cement—emphasizing the importance of the slurry concentration on penetration as the pore size decreases.

Fig. 2(*c*) shows **a** 20-mesh to 28-mesh sand **grouted** with a 20-gal slurry. About 50% of dry cement passed through the sand but the concentration within the pores of the sand was rather low. This sand was the finest into which any measurable quantity of slurry could be forced when slurries as dilute as 14.5 gal of water per sack of cement were used.

Weight, Volume, and Shrinkage of Cement Slurry

To make a comparison of the volume of slurry passing into the sand from the data of Col. 5, Table 1, it was necessary to know how much each slurry will decrease in volume as a result of the excess water in the mix. In addition to this information, it was also necessary to know the volume of the slurry per

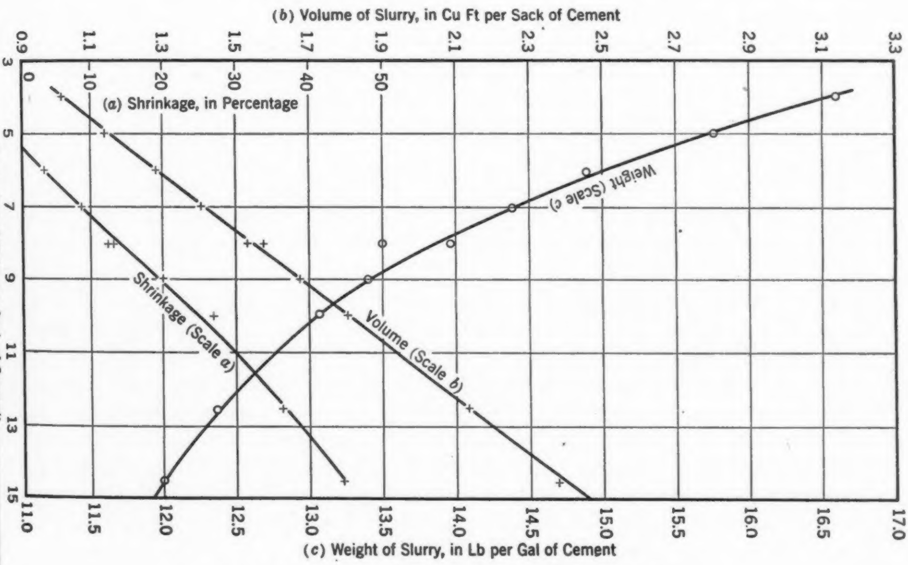

Fig. 3.—Weight, Volume, and Shrinkage of Cement Slurry

unit weight of cement in the mix to compute the amount of dry cement in the slurry above the sand prior to squeezing. To secure these data, a weighed quantity of cement and a measured quantity of water were mixed in a chemical graduate and the resultant volume was found. The results were then converted to cubic feet of slurry per 94-lb sack of cement; and these values were plotted as ordinates with the values of gallons of water per sack of cement as abscissas in Fig. 3. This curve may be considered sufficiently accurate for use on any brand of ordinary Portland cement. The values of the weight of a slurry in pounds per gallon of slurry would be of interest in field work to control the consistency of the slurry. Since a curve showing these values could be computed easily from the same data, it, too, is given in Fig. 3.

These slurries were then poured into test tubes about 5 in. high and allowed to stand for two days. The amount of shrinkage of each tube was measured

and computed as a percentage of the initial height. These values also were plotted in Fig. 3. Extrapolation of this curve to zero shrinkage would give a 5.4-gal slurry—apparently containing just the amount of water necessary to hydrolyze the cement.

Sand Permeabilities

Under the heading, "Cement Penetration Through Sands: Materials," the reason why graded sands could be used to represent natural sand formations is given. For laboratory investigations this method of securing a wide range of characteristics is very convenient; but, to correlate these data on penetration with penetrations to be expected in natural sands, a better method than mesh size is necessary to describe the sand. Since the flow of slurry into a sand would depend on the hydraulic characteristics of the sand, the permeability would be the best property with which to correlate penëtration. In truth, the penetration of a cement would depend on the ratio of the cement particle size to the sand pore size. However, the permeability is a measure of the sand pore size; and, since it is easy to determine, it is the most practical characteristic on which to base a correlation. From an academic point of view, the ratio of cement particle size to sand pore size is of interest; it is the key to further developments in the art of increasing or decreasing the cement penetration. Therefore, both qualities, the permeability and the sand pore size, were determined.

For most water-bearing sands, the simple variable-head apparatus of the type described by L. K. Wenzel, Assoc. M. ASCE, in 1942[6] will be found suitable for determining permeabilities. However, on this work sands were to be tested having permeabilities beyond the range of this instrument; and, therefore, a constant-head apparatus had to be used. A second important reason necessitating the use of a constant-head apparatus was that it furnished, with absolute certainty, data in the region of streamlined flow. To calculate the average size of pore from the flow of water through sand, Poiseuille's law may be used; but this law applies only to streamlined flow of fluids—not to turbulent flow. Consequently, it was necessary to find a way of determining whether the flow was streamlined or turbulent.

An examination of the literature shows that the well-known curve obtained by plotting the friction factor, calculated from Darcy's law, against the Reynolds number applies to flow through porous media as well as to flow through pipes. Several groups of investigators have worked on this same problem, Joseph Chalmers, D. B. Taliaferro, Jr., and E. L. Rawlins[7] obtained such a curve by computing the Reynolds number from the mean effective pore diameter. G. H. Fancher and J. A. Lewis[8] derived another curve of a similar shape by using the diameter of the average grain. With a knowledge of the mechanical analysis of the sand, the permeability and pore size, supposedly, could be computed from the latter curve.[8] Before this was done, however,

[6] "Methods for Determining Permeability of Water-Bearing Materials," by L. K. Wenzel, *Water-Supply Paper No. 887*, U.S.G.S., U. S. Govt. Printing Office, Washington, D. C., 1942, p. 59.

[7] "Flow of Air and Gas Through Porous Media," by Joseph Chalmers, D. B. Taliaferro, Jr., and E. L. Rawlins, *Transactions*, A.I.M.M.E., Vol. 98, 1932, p. 388.

[8] "Flow of Simple Fluids Through Porous Materials," by G. H. Fancher and J. A. Lewis, *Industrial and Engineering Chemistry*, Vol. 25, 1933, p. 1145.

the curve was transposed to the units used by Messrs. Chalmers, Taliaferro, and Rawlins. It was found that the two curves did not agree, and neither of the two curves could be used safely for the sand in these experiments. Nevertheless, the basic principle used by these investigators is sound; and, by obtaining such curves for the sands used in this work, the permeability and pore size of each sand could be computed.

Apparatus.—To secure the data, the constant-head apparatus, shown in Fig. 4, consisting of a metal tube 12 in. long and 1.633 in. in diameter, was used. The sand sample was packed into this tube and was held in place by 65-mesh screens on ferrule rings placed at each end of the tube. Each ring was held in position by three sharp-pointed screws fitting into sharp-recessed holes. In this manner a constant volume could be secured easily for each sample. The tube was drilled and tapped for ¼-in. cone valves for manometer taps. The inner end of each tap was machined to the curvature of the tube, and wire cloth was soldered on to prevent the passage of sand. The final assembly of the taps was made carefully so that the wire cloth was flush with the tubing.

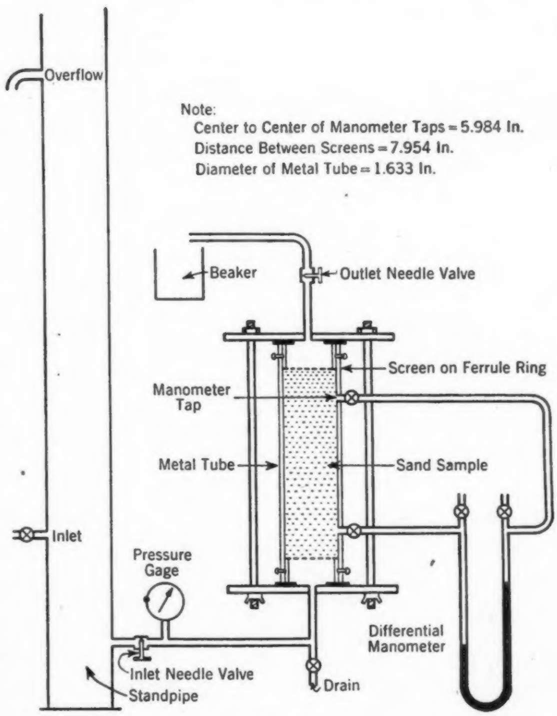

FIG. 4.—SCHEMATIC DIAGRAM OF CONSTANT-HEAD PERMEAMETER

Procedure.—A weighed quantity of sand was placed in the tube and compacted by striking the tube with a hard rubber hammer. The quantity of sand added had to be just sufficient to support the top screen in its proper position. The screen could then be tightened in place. From the weight of sand, the specific gravity of the sand, and the volume of the container between the screens, the porosity of each sample could be computed.

After the standpipe was overflowing at a steady rate, water was admitted to the sand container very slowly so as not to trap any air bubbles.

An adjustment of the outlet needle valve allowed the desired rate of flow to be obtained and, at the same time, permitted the maximum possible pressure to be maintained within the tube. This maintenance of pressure within the

tube reduced the possibility that the dissolved gases would separate from the water with a consequent air binding of the sand. The water passing out of the sand was timed and weighed to determine the rate of flow. The temperature of each run was measured to give the viscosity correction.

Notation.—The letter symbols in this paper defined where they first appear in the text, conform essentially with "Soil Mechanics Nomenclature" (ASCE *Manual of Engineering Practice No. 22*) and with American Standard Letter Symbols for Hydraulics (ASA—Z10.2—1942) prepared by a Committee of the American Standards Association, with Society representation, and approved by the Association in 1942.

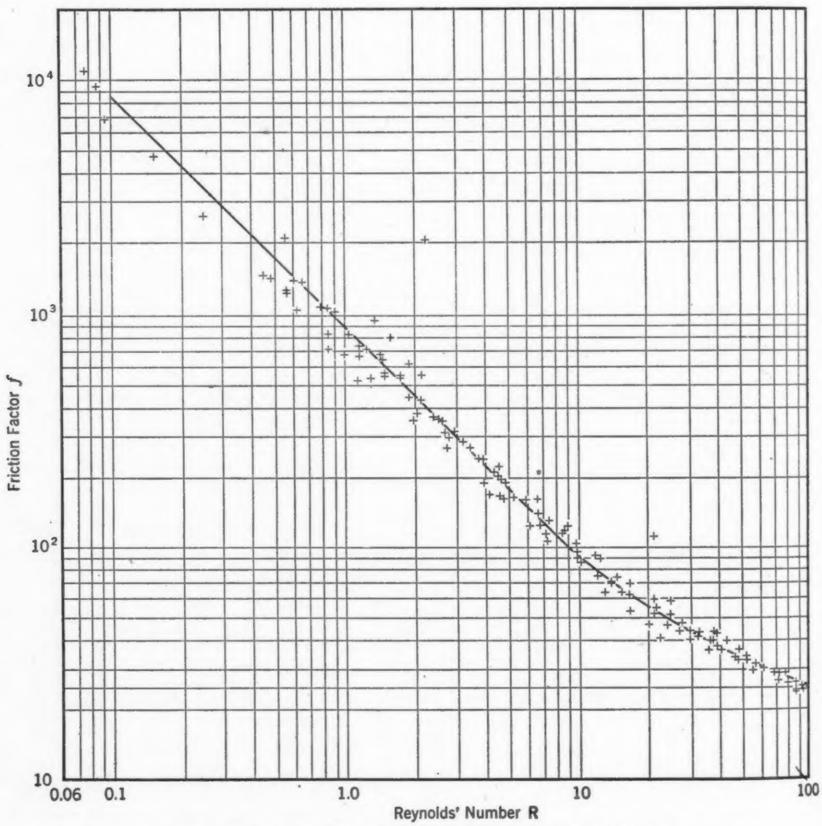

FIG. 5.—THE REYNOLDS NUMBER VERSUS FRICTION FACTOR

Data and Discussion of Results.—Six different sand sizes were tested and the value of the Reynolds number and the friction factor for each reading were computed. The plot of the two values on log-log paper gave a curve for each of the sands tested. However, all these curves were so close together that the average curve might well define them all. The plot of these two values is shown in Fig. 5.

The Reynolds number is defined as—

$$\mathrm{R} = \frac{D_g\, V\, \gamma}{\mu} \qquad (1)$$

—which is a pure number, so that any consistent set of units might well be used. If the English system is used, the values would be defined as: D_g is the diameter of the average sand grain, in feet; γ is the density of the fluid, in pounds per cubic foot; μ is the absolute viscosity, in pounds per foot per second; and V is the velocity of flow over the full area, in feet per second; that is:

$$V = \frac{B}{A_t\, t} \qquad (2)$$

in which B is the quantity of flow, in cubic feet; A_t is the cross-section area of the tube, in square feet; and t is time, in seconds. The friction factor, f, may be computed from the converted form of Darcy's law known as Fanning's formula, thus:

$$f = \frac{g\, D_g\, (p_1 - p_2)}{2\, L\, \gamma\, V^2} \qquad (3)$$

in which g is the acceleration of gravity, in feet per second per second;[9] $p_1 - p_2$ is the pressure loss due to friction, in pounds per square foot; and L is the length of core, in feet.

To determine the average grain diameter D_g, it was assumed that there was an even gradation of sizes between the two sieves used in the separation of the sand. The mean of the openings was found and used as the average grain diameter. In a case where a composite sand is used, the average grain diameter may be found by the method of Messrs. Fancher and Lewis using the formula:

$$D_g = \sqrt[3]{\frac{\sum(N\, D^3_s)}{\sum N}} = \sqrt{\frac{N_1\, D^3_{s1} + N_2\, D^3_{s2} + N_3\, D^3_{s3} + \cdots}{N_1 + N_2 + N_3 + \cdots}} \qquad (4)$$

In Eq. 4 D_s is the average diameter of the grain retained between two sieves and N is the number of such grains per gram. The values of N are given in Table 2 and were calculated from a sand density equal to 2.65 by assuming that the grains were spherical. Messrs. Fancher and Lewis proved the validity of this expression by actual microscopic count.

TABLE 2.—NUMBER OF GRAINS FOR ANY STANDARD TYLER SCREEN

Sand size (meshs per in.)	D_s, average diameter (cm)	Number (N) of grains per g
30 to 40	0.0442	8,230
40 to 50	0.0330	20,200
50 to 80	0.0227	61,900
80 to 100	0.0161	173,500
100 to 200	0.0111	530,000

Although it may at first appear incongruous to neglect the porosity completely in computing the velocity of flow (see Eq. 2), the method is found to give as good a correlation as the more complicated expression in which a porosity term is included in the velocity computation; thus:

$$V = \frac{B}{A_t\, t} \times \frac{1}{n} \qquad (5)$$

[9] "American Civil Engineers' Handbook," by Thaddeus Merriman and Thomas H. Wiggin, John Wiley & Sons, Inc., New York, N. Y., 5th Ed., 1941, p. 1323.

Furthermore, even the inclusion of the porosity in the calculation of the velocity would scarcely be more than an approximation of the true average velocity, for no exact method has yet been devised to measure the effect of the tortuous channel of flow or the effect of the many expansions and contractions.

When it became obvious that the curves for all sands could be represented by one line, it was no longer necessary to run tests on each sand. The permeability and pore size of any sand used in the tests on cementing could be computed from this curve.

In the region of streamlined flow the curve in Fig. 5 is a straight line with a slope of unity. When turbulent flow begins to occur, the curve gradually breaks away from the straight line. Since both the equation for permeability and the equation for pore size depend on measurements in the region of streamlined flow, only this part of the curve is of interest. A comparison between this curve and the one found by Messrs. Fancher and Lewis will show that the latter is somewhat higher in position. However, the curve found by the writer coincides (1) with the curve of T. H. Chilton and A. P. Colburn;[10] and (2) (when the data are converted to the proper units) with the curve of Messrs. Chalmers, Taliaferro, and Rawlins. An examination of the curve used by Messrs. Fancher and Lewis will show that there is a consistent difference in the friction factor which gives results exactly twice as large as those of the other workers. Thus, it appears that the difference may reflect an error in the computation of the friction factors by these writers.

The excellent agreement of results of data gathered by a number of different workers using widely varying granular materials and different types of apparatus indicates that this curve may be used to compute the permeability of various unconsolidated sands with a high degree of accuracy, merely from a mechanical analysis.

TABLE 3.—Sand Permeabilities and Pore Diameters

Sand size (meshes per in.)	Average diameter (cm)	Permeability (gal per day per sq ft)	Diameter of Pores (Cm)	
			Eq. 7	Corrected
(1)	(2)	(3)	(4)	(5)
4 to 8	0.3531	127,200	0.0731	0.2535
8 to 10	0.2007	41,400	0.0418	0.1452
10 to 14	0.1410	18,800	0.02805	0.0972
14 to 16	0.1080	12,100	0.02245	0.0778
16 to 20	0.0912	8,500	0.0188	0.0652
20 to 28	0.0711	5,200	0.01482	0.0514
28 to 30	0.0546	3,050	0.01132	0.0392
30 to 40	0.0442	2,015	0.00918	0.0318
40 to 50	0.0330	1,125	0.00686	0.0238

The permeabilities for the various mesh sizes of sand given in Table 3 were computed from velocity and pressure data taken to conform with the curve of Fig. 5. The coefficient of permeability k is given in units of gallons per day square foot of area for a 100% gradient. All readings were adjusted to water at 60° F. To compute the coefficient of permeability, Darcy's law is used in the form:

$$k = \frac{\mu_t}{\mu_{60}} \frac{B}{A\, t\, (h/L)} \quad \text{.....................(6)}$$

[10] "Pressure Drop in Packed Tubes," by T. H. Chilton and A. P. Colburn, *Industrial and Engineering Chemistry*, Vol. 23, 1931, p. 913.

in which μ_t is the absolute viscosity at the test temperature; μ_{60} is the absolute viscosity at 60° F; A is the total cross-sectional area of flow, in square feet; B is the volume of flow, in gallons; t is the time, in days; and h/L is the hydraulic gradient, in feet of water per foot of core length.

A comparison of the permeabilities of sand formations, secured from well logs with the permeabilities of Table 3, will give the equivalent sand size tested for cement penetration. On referring back to Table 1 for the data on the percentage of cement passed at various concentrations into this sand, the percentage of cement lost into the formation will be found. If the quantity lost is excessive, the slurry will have to be treated chemically to change the properties so that the loss may be reduced. On the other hand, if it is desired that the slurry penetrate into the sand as in the several cases previously described, the data will be of value in predicting the quantity of cement that will pass.

An idea of the range of permeabilities that may be expected in natural sands may be gathered from the paper by Mr. Wenzel:[6] Examples of sands tested show a range from 0.0002 gal to 90,000 gal per day.[11] The sand having a permeability of 90,000 was secured from a well on Long Island, New York, at a depth of about 200 ft. This high permeability is unusual, of course, but it is cited to show that such conditions may be encountered at times. Sands from wells at Salisbury, Md., with permeabilities from 252 to 1,190 are also reported by Mr. Wenzel. The author has tested several samples secured from outcrops in the vicinity of the Baltimore industrial area and has found sands with permeabilities of 1,000. Some samples taken from wells in the industrial area would have permeabilities of several thousand. One such sample was secured and tested to measure the passage of a 10-gal slurry. All the slurry passed through the sand readily. Whether this sample was representative of the sand in the well or whether its fines were washed off in the process of removal is unknown.

If highly permeable sands are encountered that might cause an excessive loss of cement, it would be necessary to find some method of plugging the pores. To attack the problem intelligently, it is first necessary to gain some idea of the magnitude of the pore opening. If the work of L. C. Graton and H. J. Fraser[12] is examined, it can be noted that the shape of a pore formed by spherical particles is far from a tubular opening. By systematic packing this shape may be varied to give a void ratio of from 26% to 48%. With natural sands, even more complex shapes may exist. Therefore, the only way to indicate the size of the pore is to compare it with the diameter of a capillary tube that has the same hydraulic characteristics as the sand pore—that is, with a straight tube which will pass the same amount of fluid with the same pressure loss as the tortuous path of the pore. Using the same data previously used to compute the permeabilities, the mean pore diameter may be calculated from Poiseuille's law; thus:

$$D_p = \sqrt{\frac{32\,\mu\,V\,L}{g\,e\,(p_1 - p_2)}} \quad \text{.......................(7)}$$

in which D_p is the mean pore diameter, in feet; and e is the void ratio.

[11] "Methods for Determining Permeability of Water-Bearing Materials," by L. K. Wenzel, *Water-Supply Paper No. 887*, U.S.G.S., U. S. Govt. Printing Office, Washington, D. C., 1942, p. 13.

[12] "Systematic Packing of Spheres—With Particular Relation to Porosity and Permeability," by L. C. Graton and H. J. Fraser, *Journal of Geology*, Vol. 43, No. 8, 1935, Pt. 1, p. 785.

The values obtained by using Eq. 7 are presented in Col. 4, Table 3. However, for the correct application of Poiseuille's law to flow through porous media, it is essential to make a number of corrections in the data. The law applies only to the pressure lost in the viscous flow of a fluid through a straight circular section. In a porous medium the cross section of the flow channels is not circular but is irregular. The path is not straight but is sinuous with many alternate enlargements and contractions. C. S. Slichter,[13] using spherical particles, showed that the path of fluid flow was from 1.2 to 1.5 times the length of the core and that the average velocity of the fluid was about 1.8 times the velocity calculated from the rate of flow through the area of the voids. Messrs. Chilton and Colburn[10] claim that only about 20% of the pressure loss is caused by the friction of viscous flow and that the loss caused by the rapid expansion and contraction of the fluid is responsible for the major part of the remainder. Making these corrections on the length of path, the velocity, and the pressure would increase the value under the radical in Eq. 7 by about twelve times and the pore diameter by the square root of 12. The pore diameters corrected by this amount are given in Col. 5, Table 3. These corrected values are much closer to the size of openings that can be observed microscopically in core cross sections than are the uncorrected values.

The cement used in this work, when tested for fineness, gave 96% passing the 200-mesh sieve (0.0074-cm opening). The 28-mesh to 30-mesh sand was the largest size which did not pass a 14.5-gal slurry. Comparing the 96% cement size with the pore size of the sand shows the average pore to be five times the cement size. Taking into account the fact that the average diameter of the pore is being compared, and not the minimum constricted diameter, it appears that the cement ceases to pass into the pore when the pore becomes small enough for two or three particles to bridge across it.

From this evidence it appears that the passage of cement into a sand is limited by the ratio of the cement particle size to the sand pore size. A sand having a permeability of less than 3,000 gal per day per sq ft will not allow any measurable quantity of cement to pass into it regardless of the squeeze pressure or of any other factors.

Slurry Filtration on Fine Sands

Thus far in the paper, the description has dealt only with factors affecting the penetration of cement slurry into sands. The next feature that requires attention is the problem of placing a cement slurry against a fine sand. The greatest percentage of wells in the Baltimore area probably penetrate sands with permeabilities less than 3,000. Hence, the cement particles will not penetrate into the sand but will build up a filter cake as the water is forced from the slurry. If the pressure gradient between the slurry and sand is high, it is quite possible that the water may filter from the slurry more rapidly than the slurry is being pumped into the well. In such a case, the slurry cake may clog the inlet tube, stop the pump, and leave an incomplete job.

[13] "Theoretical Investigation of the Motion of Ground Water," by C. S. Slichter, 19th Annual Rept. U.S.G.S., U. S. Govt. Printing Office, Washington, D. C., Vol. 2, 1898, p. 305.

To solve this problem, the effect of the pressure, the rate of pumping, the slurry concentration, and the sand permeability on the rate of formation of the slurry cake must be known. By correlating all these factors it may be possible to choose the correct slurry concentration, pumping rate, and squeeze pressure to insure a satisfactory completion of a grouting job for any well. When a cement slurry is placed in a well and the pressure in the slurry is greater than the pressure in the sand formation, the water will be squeezed out of the slurry leaving a filter cake on the wall of the well. In effect, this is a giant filter press and it is possible to simulate this press with a laboratory model. By starting with the simple concept of considering the filtration rate as proportional to the pressure divided by the resistance and considering the resistance as being made of two parts, the resistance of the filter cake and the resistance of the filtering medium, it is possible to arrive at a filtration equation. This equation will include all the known factors that will affect the filtration process. By measuring the constants that depend on the slurry and the filtering media, a solution of the equation will yield the result of a change in any one of the variable factors previously discussed. In this manner a precise measure of each of the variables may be obtained.

A good treatment of filtration equations and their derivations has been published by W. H. Walker, W. K. Lewis, W. H. McAdams, and E. R. Gilliland.[14] For the cement grouting of a well, a constant-pressure filtration equation will best fit the conditions usually existing in the field; and, therefore, the following equation for constant-pressure filtration[15] will be chosen for investigation:

$$\frac{p_f\, t}{W_t/A_f} = \frac{r''\, B_c}{2\,a}\,\mu\, a\, p^{k}{}_f \times \frac{W_t}{A_f} + r'\,\mu\, p^{m}{}_f \qquad (8)$$

in which p_f is the total filtering pressure on the cake, in pounds per square inch; W_t is the total weight of filtrate from the slurry, pounds to time t; t is the total time of operation, seconds; A_f is the total area of the filtering surface, in square inches; r' is a constant coefficient depending on the resistance of the filter bed; r'' is a constant coefficient depending on the specific resistance of the cake; B_c is the volume of the cake as it collects on the filter, in cubic inches per pound of filtrate; a is the concentration of the slurry, in pounds of solids, per pound of filtrate greater than that necessary to make a 5.5-gal slurry; m is the coefficient of plugging for the filter media; μ is the absolute viscosity of the filtrate in centipoises; k is the coefficient of compressibility of the filter cake; and m is the coefficient of plugging of the filter media.

The value of the concentration factor a was computed quite empirically. Ordinarily, the factor a is defined as the pounds of solids per pound of filtrate; but, when this definition was applied to the data obtained in the tests, it was impossible to secure the same results with different slurry concentrations. In the experiments on the shrinkage of slurries, it was found that 5.4 gal of water per sack of cement was the quantity of water required to just hydrolyze the

[14] "Principles of Chemical Engineering," by W. H. Walker, W. K. Lewis, W. H. McAdams, and E. R. Gilliland, McGraw-Hill Book Co., Inc., New York, N. Y., 1937, p. 342.

[15] *Ibid.*, p. 347.

cement. If the value of a is based on the amount of water in excess of this quantity, which is the quantity causing the fluidity of the slurry, the a-value secured would be comparable to that which is ordinarily used for solids that do not react chemically with the filtrate. Instead of using 5.4 as the basis, the value was rounded to 5.5. This hypothesis applied to the data was found acceptable. Since the filtration equation itself is solved empirically, this definition of a is satisfactory.

In the laboratory, only the values of $\left(\frac{r'' B_c}{2\,a}\right)$, k, r', and m (which will be constants in Eq. 8) must be measured to reduce the equation to a relatively simple form. It is then an easy matter to measure the effect of any one variable on the filtration process by changing the variable and solving the equation for the amount of filtrate lost. When the amount of filtrate lost is known, it is necessary only to subtract this quantity from the amount pumped in to determine the concentration of the remaining slurry.

In the filtration formula (Eq. 8), the term $\left(\frac{r'' B_c}{2\,a}\,\mu\,a\,p^k{}_f \times \frac{W_t}{A_f}\right)$ is a measure of the resistance of the filter cake. For a given slurry the values of $\left(\frac{r'' B_c}{a}\right)$ and of k should be constant. In practice, however, experiments show that, even with an incompressible material such as a "kieselguhr" slurry, the values vary slightly with the pressure and with the concentration of the slurry. With compressible materials the variation is often considerable.

The last term of Eq. 8 $(r'\mu\,p^m{}_f)$ is a measure of the resistance of the filter medium. Again, the terms r' and m should be constant for a given filter medium; but in practice they usually vary somewhat with the pressure and with the slurry concentration. The resistance of the filtering medium is not the resistance of the medium when no slurry is present, but rather the resistance after the solid particles have plugged the pores of the medium. Thus, the constant terms change in value with changes in the slurry and in the pressure.

Materials.—The cement and sand used in this work were the same as those described previously under the heading, "Cement Penetration Through Sands: Materials." The coarsest sand used for a filter medium was the 30-mesh to 40-mesh material, with a permeability of 2,015 gal per day per sq ft, and the finest material tested was a blended sand of from 30-mesh to smaller than 200-mesh. This sand had a permeability of 11.6 gal per day per sq ft. The other two sands used had permeabilities of 529 and 132.

The permeability of each of the sands was computed from the Reynolds number versus friction factor diagram, by the method previously outlined, and checked by a variable-head permeameter. The Reynolds number versus friction factor diagram was found to give excellent results.

Apparatus.—The apparatus used for these tests was the same as that used for the tests of cement penetration. The schematic diagram of the apparatus is shown in Fig. 1.

Procedure.—The procedure used in testing followed lines similar to those used in the tests on cement penetration. The pressure forced the filtrate from

the slurry into the sand, replacing the water already there. The excess water passed out of the sand and was caught in a chemical graduate. A knowledge of the total proportion of filtrate that could be expected from the slurry was gained from experience, and an attempt was made to catch about one sixth of the filtrate in each of six graduates. The filling of each graduate was timed by a stop watch. In this manner it was possible to secure six points for each run so that a curve could be drawn. At the end of each run the apparatus was backwashed with water to remove all the slurry in the tubing.

Data and Discussion of Results.—Eq. 8 is a formula of the form $y = m\,x + b$ in which W_t varies with t as the pressure is held constant. If the data fit this equation, a plot of the values of $\dfrac{p_f\,t}{W_t/A_f}$ against the values of W_t/A_f should give a straight line of the type shown in Fig. 6. The slope of this straight line will be $(r''\,B_c/a)\,(\mu\,a/2)p^k{}_f$ and the intercept on the ordinate at $W_t/A_f = 0$ will be

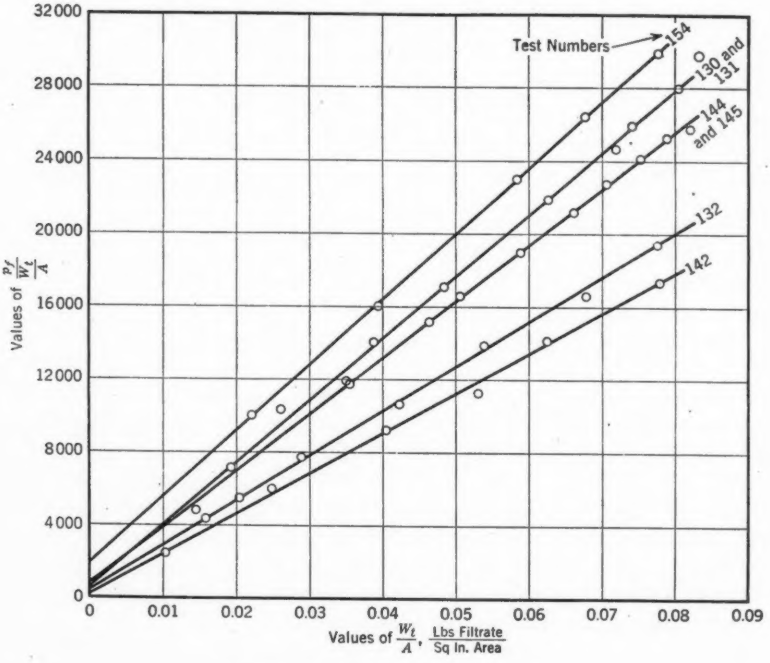

FIG. 6.—GENERAL FILTRATION CURVES

$r'\,\mu\,p^m{}_f$. When the value of the slope of the line is divided by $\mu\,a/2$, the remainder will be $(r''\,B_c/a)p^k{}_f$. A plot of the values of $(r''\,B_c/a)p^k{}_f$ against the values of p_f on log-log paper will give a straight line with a slope of k. Such a plot is shown in Fig. 7. The intercept of this line on the ordinate at $p_f = 1$ will be the value of $(r''\,B_c/a)$. This method of analysis furnishes the values of the two constants $(r''\,B_c/a)$ and k for the filtration equation.

By a similar procedure it is possible to evaluate the constants r' and m. The values of $r' \mu p^m{}_f$, secured from the first plot of the type shown in Fig. 6, when divided by the viscosity, will give $r' p^m{}_f$. A plot on log-log paper of the values of $r' p^m{}_f$ against the values of p_f will give a straight line with a slope of m. A plot of this type is shown in Fig. 8. The intercept of the line on the ordinate at $p_f = 1$ will be the value of the constant r'.

When the constants for Eq. 8 have been found in this manner, their substitution will reduce the equation to a form which is relatively simple to handle.

The first experiments were conducted with a 30-mesh to 40-mesh sand and a 14.5-gal slurry. This very dilute slurry was chosen for the first tests because it was very fluid and easy to handle. Pressures from 5 lb per sq in. to 45 lb per sq in. were used in making the series of tests. For each run the pressure was kept constant and six readings on the quantity of filtrate with the corresponding time were taken. From these data, the series of curves of the type shown in Fig. 6 were drawn.

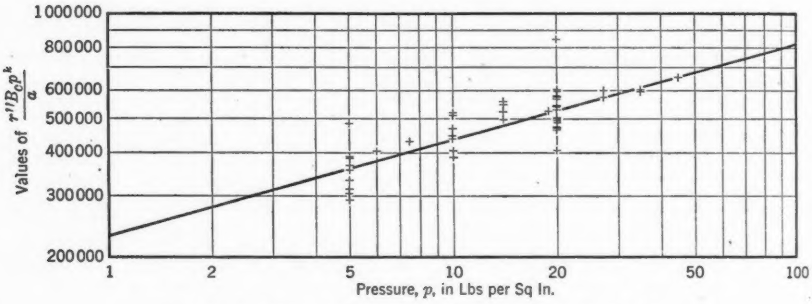

FIG. 7.—FILTRATION CURVES FOR THE DETERMINATION OF k

To determine the effect of slurry concentration, 20-gal and 8-gal slurries were also tested on the same sand. When the slopes of the resulting lines were plotted in Fig. 7, it was found that the 20-gal slurry gave a line parallel to, and slightly below, the 14.5-gal slurry line. The 8-gal slurry gave a line parallel to both of the other two lines but slightly above them. Nevertheless, all these lines were sufficiently close together so that the average line could be used to represent any one. Furthermore, the plugging resistance given by the last term of Eq. 8 tends to counterbalance this difference. Thus, the final result will be reasonably accurate in any of the cases.

To secure the constants for the term involving the resistance of the sand, the intercepts on the ordinate at $W_t/A_f = 0$ of the original plot (Fig. 6) were divided by the value of the viscosity and plotted against the pressure readings, giving a series of curves of the type shown in Fig. 8. The intercept on the ordinate at $p_f = 1$ is the value of r' and the slope of the lines is the value of m.

The line for the 20-gal slurry is very nearly parallel to the line for the 14.5-gal slurry but is slightly above it. The value of m, therefore, is the same in each case; but the value of the intercept, r', is higher in the case of the 20-gal slurry. The 14.5-gal slurry line may be taken as representative of each case

because the slight change in the constants of the slurry resistance term and the sand resistance term balance each other.

The next important step is the determination of the change in the resistance of the sand as the permeability is decreased. For this work three other sands with permeabilities of 529, 132, and 11.6 gal per sq ft were used. A 14.5-gal slurry was squeezed against each one and the filtration curves were drawn. The slopes of the "General Filtration Curves," as shown in Fig. 6, were corrected by dividing by $N/2$ and multiplying by a to obtain the term $\frac{r'' B_c p^{k}{}_{f}}{a}$. These corrected slopes are also included in Fig. 7 for the determination of $r'' B_c/a$ and k. The points define the same quantities as those previously obtained so that the cake resistance term is not affected. The filter media resistance, on the other hand, is changed considerably. The value of m is unchanged, but the value of r' increases with a decrease in the permeability. This change in r' was plotted on various types of graph paper, and it was found that, on semilogarithmetic paper (Fig. 9), it produced very nearly a straight line between the permeabilities of 2,000 gal per day per sq ft and 100 gal per day per sq ft. Somewhere between the permeabilities of 100 and 0 the straight-line relationship ceased to hold and the curve became asymptotic to the line of 0 permeability. Only one point in this region was found—the point at a permeability of 11.6. Further experience may indicate that a closer definition of this line is desirable, but for the present this paper will limit its scope to the part of the curve above a permeability of 100.

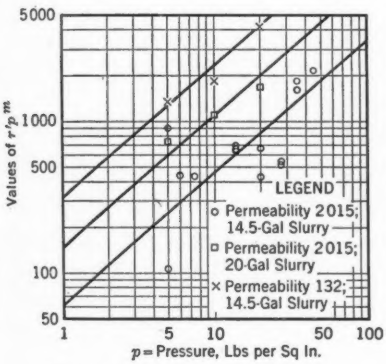

FIG. 8.—FILTRATION CURVES FOR THE DETERMINATION OF m

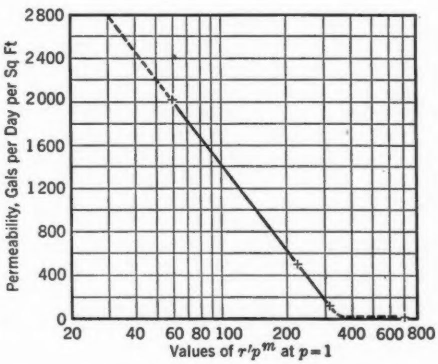

FIG. 9.—EFFECT OF PERMEABILITY ON THE VALUE OF r'

The straight-line section of the curve probably can be extended to a permeability of about 3,000. Two runs on a sand with a permeability of 5,200 were made, but about 4% of the cement passed through the sand. The filtration equation (Eq. 8) was not applicable, and no further tests in this region were made.

The slope of the curve of Fig. 7 gives a value of 0.278 for k and the intercept on the ordinate gives a value of 230,000 for $r'' B_c/a$. From Fig. 8, the slope of the line gives a value for m of 0.88. The intercept on the ordinate which is r' varies with the permeability and may be found from Fig. 9. For a sand with a permeability of 2,015, the value of r' is 60 and Eq. 8 becomes:

$$\frac{p_f\, t}{W_t/A_f} = 230{,}000 \left(\frac{\mu\, a}{2}\right) p^{0.278}\, (W_t/A_f) + 60\, \mu\, p^{0.88} \quad\ldots\ldots\ldots(9)$$

To determine the importance of the various factors of Eq. 9, a typical hypothetical case of a cementing job may be assumed and the values determined. For example, assume that a 14.5-gal slurry is to be squeezed against a 10-ft salt-water formation. The hole is 18 in. in diameter and is cased with a 14-in. casing. The slurry temperature is 68° F, and the sand has a permeability of 2,015. It is desired to know how long it would take to squeeze enough water out of the slurry to leave a 5.5-gal slurry behind the casing if 30 lb per sq in. of pressure is applied. It was found that 50.8 sec would be required. The following example illustrates the solution of the problem:

The area of the sand formation is: $A_f = \pi \times 18 \times 10 \times 12 = 6{,}780$ sq in.; the viscosity of the water, $\mu = 1.005$ centipoises; and the slurry concentration factor, $a = \dfrac{94}{(14.5 - 5.5)\; 8.33} = 1.252$ lb of solids per pound of filtrate. The definition of a is the pounds of solid per pound of filtrate in excess of 5.5 gal of water per sack of cement. For this calculation a is the ratio of the weight of a sack of cement to the weight of 9 gal of excess water. The volume to be filled outside the casing $= \dfrac{\pi}{4}\left[\left(\dfrac{18}{12}\right)^2 - \left(\dfrac{14}{12}\right)^2\right] 10 = 7$ cu ft.

From Fig. 3, a 5.5-gal slurry occupies 1.22 cu ft per sack of cement. To fill 7 cu ft will require $7/1.22 = 5.74$ sacks of cement. A 14.5-gal slurry occupies 2.40 cu ft per sack of cement; and, therefore, the volume of filtrate which must be seueezed out is $(2.40 - 1.22)\; 5.74 = 6.76$ cu ft, or 423 lb. Therefore, $W_t/A_f = 423/6{,}780 = 0.0624$ lb per sq in.

The value of r' from Fig. 9 is 60. Consequently, $\dfrac{p_f}{W_t/A_f} = 230{,}000 \left(\dfrac{\mu\, a}{2}\right) p^{0.278}{}_f \left(\dfrac{W_t}{A_f}\right) + 60\, \mu\, p^{0.88}{}_f$; $\dfrac{30\, t}{0.0624} = 230{,}000\; \dfrac{(1.005 \times 1.252)}{2}\; (30)^{0.278}$ $(0.0624) + 60\,(1.005)\,(30)^{0.88} = 23{,}200 + 1{,}208 = 24{,}408$; and $t = 50.8$ sec.

If the sand permeability were assumed to be 11.6 instead of 2,015, the time required would be 77.6 sec. All values shown in the solution of the foregoing problem would remain the same except the value of r'. From Fig. 9, the value of r' would be 700. A sand with the permeability of 11 would be considered as a low permeability material. On the other hand, a permeability of 2,000 is very high. For comparison with these values, it may be stated that, to be considered a good aquifer, a sand should have a permeability of 250 or higher.

From these calculations it is evident that, within this range, the permeability of a sand is a factor of small importance in the filtration of a cement slurry.

Clay beds would probably absorb the filtrate so slowly that the area of clay surfaces may be neglected in the computations. Even if this were done, the time of filtration in most cases still would be so short that it would make little difference in the type of equipment necessary to place the slurry.

The concentration of the slurry is a factor of great importance when using either the tubing method or the casing method of cementing. If a slurry with a low-water content is used, the water in the slurry may filter out more rapidly than the pump can place the slurry. In this case, only a few feet of slurry may be placed behind the casing before the concentration increases to the point where it is no longer a fluid but a solid. The outlet of the grout pipe will then clog and the filter cake will build up within the pipe stopping the pump.

It becomes obvious that the pump used to place the grout must have a capacity sufficiently high to place the required volume of slurry before the concentration increases to the point where the slurry will clog the grout tube. In the previous example, a pump capacity of at least 124 gal per min would be necessary. The pump must place 13.8 cu ft of slurry, but the final volume is 7 cu ft. It should also be noted that doubling the height of hole to be cemented will double the pump capacity required, because the time computed by the filtration equation will still be the same.

The oil well industry learned long ago that high-capacity pumps were essential for the full completion of a grouting operation. Companies were formed which specialized solely in the cement grouting of wells. Operators learned that special equipment was necessary to grout a well successfully and called upon these concerns to furnish the service. In much of their advertising, great emphasis has been placed on the high-capacity, high-pressure pumps.

High pressures are neither necessary nor desirable for squeezing the excess water from the slurry. Even though high pressures are used in the oil well industry for squeeze cementing, it must be remembered that the total pressure in itself does not force the water from the slurry, but rather the pressure differential between the slurry and the sand formation. In the oil well industry, formation pressures of more than 1,000 lb per sq in. are fairly common. Before the slurry can be pumped into the well, the formation pressure as well as the friction loss of flow through the grout line must be overcome. For this reason, the pump pressure must always be higher than the effective squeeze pressure. From the example cited, it may be noted that a pressure differential of 30 lb per sq in. is sufficient to force the water from the slurry very rapidly. A low pressure differential, which increases the allowable time for pumping, would be more economical.

For an intelligent use of the filtration equation, the limits within which it applies must be understood. The equation is derived for a condition in which the slurry has immediate access to all parts of the filter medium, and the area of the medium does not change with time. Therefore, the equation can be expected to give accurate results when grouting is done by the principles of squeeze cementing. In this case, the salt-water formation is located, and the casing opposite the formation is bullet perforated. The slurry has immediate access to all parts of the formation through the many holes and is prevented

from passing too far beyond the formation by cavings around the casing. Under these circumstances, it would be desirable to use a high concentration of slurry to be certain that the slurry cake forms rapidly and that the slurry does not pass down the hole away from the formation.

For the grouting of a long section of casing, Eq. 8 cannot be used. With this operation, the slurry will rapidly form a thick cake around the foot of the casing. If the pumping rate is low, the grout pipe will clog; but, if the rate is high, the velocity of flow may be sufficient to scour the cake around the casing and to keep the grout flowing up the hole. However, in its path upward the slurry will lose water to the adjacent cake so that, when it arrives at the top of the column, it will be thicker than the initial concentration. The value of a in the equation will then be different, as will also the value of W_t/A_f, because of the change in area with time.

Summary

The contamination of underground fresh-water aquifers by salt-water infiltration has become a serious problem in the Baltimore industrial area and other areas. The salt water is leaking into both producing and abandoned wells from overlying formations into the fresh-water strata. This leakage generally occurs through the space between the well casing and the borehole and may be stopped by filling this space with cement slurry.

When a cement slurry is placed in a well in contact with a porous formation, the water in the slurry will be filtered from it when the pressure differential is toward the sand. If the coefficient of permeability of the sand is less than 3,000 gal per day per sq ft, the pores of the sand will be too small to allow the cement grains to enter. If the permeability is greater than this quantity, some of the cement may pass; but, since the water in the slurry may filter at the same time, the entire quantity of cement will not always move into the formation.

When a sand has a permeability of less than 3,000, the pressure will force the water out of the slurry leaving a filter cake upon the sand surface. To give an exact description of the conditions at any instant during the filtration process, the filtration equation for cement slurries was developed.

A study of the sands having permeabilities greater than 3,000 showed that three factors mainly controlled the amount of cement penetrating into the formation:

(1) The ratio of the pore size to the cement particle size;
(2) The slurry concentration; and
(3) The manner in which the pressure was applied.

Calculation of the pore diameters showed that the pore of the largest sand which would not permit the passage of a cement slurry was small enough so that two or three cement particles could form a stable bridge across the opening. No amount of pressure could be expected to force the cement into this sand. When the pores are larger, some of the cement will pass into the sand. If the slurry is thin, all the cement may penetrate; but, if the slurry is thick, the

water may filter from it too rapidly to carry along all the cement particles and a filter cake will result. By using this principle, a slurry cake may be formed on a sand with a permeability as high as 127,000.

The speed with which the pressure is applied will determine the amount of cement that will penetrate. If the pressure is applied instantaneously, a greater quantity will pass than would pass if the same pressure were applied slowly. With a slow rate of pressure application, the slurry may cease to penetrate long before the maximum pressure is reached, so that high-pressure differentials are of small importance in such a case.

Under no circumstance is there any justification for using a blanket rule for the selection of the slurry concentration as has been attempted by many writers. In some cases the slurry may be very dilute, the excess water serving only as an agent to carry the cement into position, after which it is squeezed out and the cement particles are compacted. The initial slurry concentration will have no bearing on the volume, strength, or permeability of the final product. In other cases, it may be desirable to grout off a given formation without allowing the slurry to spread over other strata. In these circumstances, it would be desirable to use a thick slurry and a high squeeze pressure.

Acknowledgments

The investigations reported in this paper were made in the laboratories of The Johns Hopkins University at Baltimore. The writer is indebted to Abel Wolman, M. ASCE, professor of sanitary engineering, The Johns Hopkins University, for advice and general supervision in carrying out the investigation. The writer is also indebted to Robert R. Bennett and R. R. Meyer, Geologic Survey, United States Department of the Interior, for information on cementing practices in the Baltimore area and in the western oil fields. Special appreciation is due to John C. Geyer, associate in civil engineering, The Johns Hopkins University, for suggesting this problem.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

LANDSLIDE INVESTIGATION AND CORRECTION

Discussion

BY RUSH T. SILL, EARL M. BUCKINGHAM, AND ALFRED V. BOWHAY

RUSH T. SILL,[14] ESQ.[14a]—An extremely well-presented treatment of the causes of landslides and their correction is contained in this paper.

The writer had the opportunity to study and suggest corrective measures in a slide in which the movement took place along faults occurring in a large mass of homogeneous rock.

Water over a large surface area percolating through the soil entered the fault and softened the gouge material transforming it into softened lubricated mass on which the rock moved.

The spillway cut at the El Capitan Dam built by the City of San Diego, Calif., on the San Diego River, as a part of its water collecting and storage system, was made through the much faulted, fractured, and altered granitic rock which forms the north abutment of the dam. Along lines of rock weakness, fault, shear, and fractured planes, weathering had progressed, developing near the surface the boulder type of weathering so common to granite. The results of this type of weathering are boulders of almost fresh, unaltered granite surrounded by weathered and altered granite less durable than the boulders.

North of the dam area, in the spillway cut about opposite the west end of the weir is a prominent fault striking north 16° east with a dip of 69° to the south. Some distance below the top of the hill, this fault had been cut by a steep westerly dipping shear plane, and Fig. 15 shows a slide of several hundred tons of the crushed material in the fault zone. This slide occurred soon after the excavation in the spillway cut had progressed below the westerly dipping shear plane.

A mass of rock estimated to be 150,000 cu yd rested on a westerly steeply dipping fault plane which extended southeasterly along the spillway cut from this fault on the north to a small gulley formed on a shear zone to the south end of the spillway cut and east 400 ft to the first prominent shear planes strike

NOTE.—This paper by Hyde Forbes was published in February, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1946, by T. W. Lambe; and September, 1946, by Jacob Feld, George S. Harman, and Robert F. Legget.

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[14a] Received July 1, 1946.

north 58° west dipping steeply to the north. The wedge-shaped block of ground was left unsupported on the lower or big end of the wedge by the excavation of the spillway (see Fig. 15). During construction, in the winter wet season, this block of ground moved approximately 6 in. down the westerly dipping shear plane, opening a series of almost continuous cracks over the crest of the ridge between the small gully formed on the north fault and the gully formed along the south shear plane.

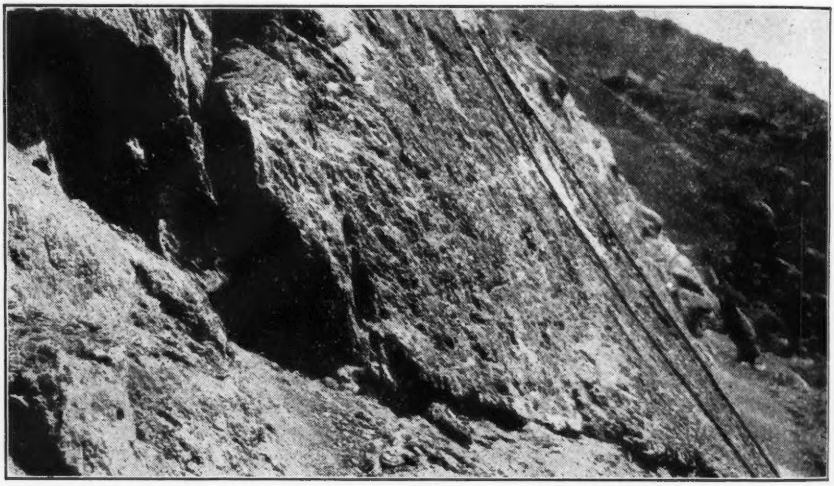

FIG. 15.—VIEW OF SLIDE, NORTH ABUTMENT, EL CAPITAN DAM, SAN DIEGO, CALIF.

These cracks in the surface soil are from 2 in. to 4 in. wide and some of them can be followed for a distance of from 40 ft to 50 ft. Continued slippage would have resulted in the movement of this 150,000 cu yd into the spillway cut. Its subsequent removal would have been a very expensive operation. Had the slide occurred and the spillway been plugged during a flood, overtopping of the dam might have resulted.

It was recommended that this block of ground be protected by well-constructed ditches to keep the water from entering this fault plane to lubricate it with resulting possible further movement.

The El Capitan Dam is in a dry section of Southern California, and further movement of this block of ground has been arrested.

The following investigations were instigated by the contractors to classify the material in the spillway cut and to confirm the surface indications as to the cause of this movement.

Crushing, specific gravity, and percolation tests were made on samples of the partly decomposed granite surrounding the boulders. This was done to establish the classification into which the material fell—solid rock (ledge rock in place that cannot be loosened except by wedging, barring, or blasting); detached masses of solid rock more than 1 cu yd in volume; or earth, or overburden. Samples were taken from seven sections spaced approximately at

equal intervals along the north face of the spillway, none of which were free of fracture lines and thin gage seams which greatly reduced their strength. For permeability tests, samples of rock were cut to a thickness of 4 in. These were placed in a circular side form composed of galvanized sheet iron; then a mortar composed of one part of Portland cement and two parts of sand was puddled around the stone, which gave a finished specimen in the shape of a disk 4 in. thick and 8 in. in diameter. The metal side forms remained on the disks during the tests. The specimens were placed in a moist room as soon as they were molded to insure proper curing of the mortar.

The disks were placed for testing between cast-iron top and bottom pieces of holders to bring water under pressure in contact with the upper surface, and to retain for measuring all water passing through the disk.

These castings were made to make contact or bearing with the disk for a width of 1 in. around the circumference; a small chamber for water was left both above and below the disk; the water was free to pass through the center of the 6-in. disk; and the castings were drawn together by tightening the six bolts in the flanges.

The upper cap was provided with a water gage and air vent; the lower cap had an outlet valve and an outlet tube leading to a glass tube graduated to cubic centimeters.

In testing, water was poured into the lower chamber; outlet valves were filled and no air was present. The outlet valve was closed so that any water passing through the disk was caught in the graduated tube. Water was then admitted to the upper cap, excluding all air. The reservoir was so arranged that the head of water was kept constant throughout the test.

TABLE 3.—Average Leakage Rate Through Typical Samples

Flow, in cubic centimeters	Specimen 1B	Specimen 4C	Specimen 5A
For a head of 80 in.	206	2,967	388
Per hour	824	11,868	1,552
Per day	19,776	284,832	37,248
Per sq in. per day	695	11,508	2,763

Readings of the water level in the container were taken at 15-min intervals, and the amount of leakage calculated. The results of these tests (Table 3) show a porous condition of this weathered granite to depths of between 200 ft and 250 ft. Near the surface percolation rates of 1,149 ft per yr were obtainedfrom sample 4C.

In making the tests on voids in rock fragments, the rock was first crushed into particles passing a No. 4 screen. The sample was poured loosely into a steel cylinder 6.17 in. in diameter and 7.93 in. high which was filled to about ½ in. below the top. A steel piston was placed on the rock fragments and brought to level bearing without pressure except its own weight; the specimen was placed in the testing machine and successive loads applied. After each load was applied and the piston had come to rest, measurements were taken of the height of the material in the steel cylinder. The increase of the load on the piston caused the rock material in the cylinder to be compressed; from the heights of the material in the cylinder, the volumes of the material at various loads were calculated; and the weights of the material per cubic foot were then calculated from the actual weight of the samples used. The apparent specific

gravity of each sample (see Table 4) was determined by the "Standard Method of Test for Approximate Apparent Specific Gravity of Fine Aggregate" of the American Society for Testing Materials (A.S.T.M. C—68—30).

TABLE 4.—CHARACTERISTICS OF FINE MATERIAL

Description	Sample 1A	Sample 5C	Sample 6B
Specific gravity (approximate, apparent)	2.39	2.76	2.76
Percentage of Voids, Under Loads (Lb per Sq Ft) of:			
Piston only	44.46	44.16	43.54
2,500	44.76	43.12	42.85
5,000	43.74	42.50	42.22
7,500	42.93	41.91	41.74
10,000	42.18	41.30	41.32
12,500	41.49	40.89	40.99

The percentage of voids under each condition was determined according to the "Standard Method of Test for Determination of Voids in Fine Aggregate for Concrete" of the American Society for Testing Materials (A.S.T.M. C—30—22).

The soft decomposed granitic material from the spillway cut disintegrates rapidly when excavated and placed in the waste pile. In order to test the percentage of voids and apparent specific gravity of the material in the waste pile, parts of the rock samples 1A, 5C, and 6B were crushed to pass a $\frac{1}{4}$-in. opening. The compaction, under 12,500 lb per sq in., equal to a height of dump of 100 ft, varied from uniform grading of the crushed material, as shown in the screen tests. Grading of stone voids in the fragments before testing was made on all the samples from the north spillway cut, the results of sample No. 1 being as follows:

Sieve No.	Individual sieve	Cumulative
200	6.3	6.3
100	4.2	10.5
80	2.8	13.3
50	7.1	20.4
40	5.6	26.0
30	8.5	34.5
20	12.8	47.3
16	7.4	54.7
10	18.3	73.0
8	7.7	80.7
4	19.3	100.0
	100.0	

The weight per cubic foot was made on samples from the spillway cut as follows: Sample 5B, 136.66; sample 1B, 160.99; and sample 5C, 166.61.

The compression tests (see Table 5) were made on all the samples. Rocks with crushing strength of 388 lb per sq in., etc., represent material that can be excavated without blasting, but in quarrying operations is drilled and blasted for economy and speed of operation.

TABLE 5.—SELECTED TESTS FOR TOTAL COMPRESSIVE STRENGTH OF STONE

Strength	Sample 4B	Sample 1B	Sample 5C
Pounds	38,220	38,220	10,380
Pounds per square inch	1,333	2,389	353

Where nests of boulders and hard-rock ribs are interspersed in this decomposed granite, it is necessary to drill and blast before excavating by a steam shovel, and should be paid for as rock.

EARL M. BUCKINGHAM,[15] Assoc. M. ASCE.[15a]—A very practical approach to a difficult problem is evidenced by the data in this paper, and it is not the intent of the writer to criticize the author for his excellent presentation. Rapid progress has been made in soil mechanics in recent years; much has been learned that is of value in slide investigation; but the fundamental problem is still one of determining the structure of the hill and the nature of the materials involved. Mathematical analysis has its place, particularly in dealing with slides that cannot be drained. However, in most cases the problem is simply one of locating and removing water. If this can be done, the increase in the strength of the soil will nearly always be more than sufficient to insure stability, and a stress analysis becomes merely an interesting academic exercise. No slide investigation is complete without a thorough determination of underground conditions, but it is frequently possible to simplify the boring program and materially reduce its cost.

Topographic Shape.—The landslide is a natural phenomenon, and one of the processes by which topographic forms are sculptured. An experienced topographer can frequently study the shape of the slide and the adjacent hillside and predict with considerable success the underground conditions that will be discovered by borings.

A long narrow shape, with movement approximately parallel to the original surface, indicates a shallow source of water, and a shallow bedrock which is also roughly parallel to the surface. (In this connection the writer broadens the term bedrock to include a firm and impervious clay.) Although it is easy to draw conclusions after the result is known, the St. Mary's Playground slide in San Francisco, Calif., appears to be typical of this class.

A broad sweeping curve at the head of the slide indicates a deep source of water, particularly if the outline of the slide is poorly defined and formed by a number of parallel cracks. This condition may be accompanied by a distinct rotary motion of the slide mass. The McKillop slide, in Oakland, Calif., shown in Fig. 16, falls in this class. The extreme breadth of the slide indicates either an unusually broad sheet of water or several more or less independent channels. At least four separate deep channels were located by borings. The small apex opposite the north arrow indicates an independent shallow channel. Shallow water was found in the slide mass, and opposite this point in the next street to the west, but the connection was not followed through private property to the head of the slide.

The almost semicircular outline of the head of the Parker Avenue slide (San Francisco) indicates a deep and fairly concentrated source, located near a luxuriant growth of shrubbery about midway between the "5" of the 375-ft contour and the head of the loop of the 350-ft contour in Fig. 2. The depth below the original surface is confirmed by the extent of the movement that has

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[15a] Received June 12, 1946.

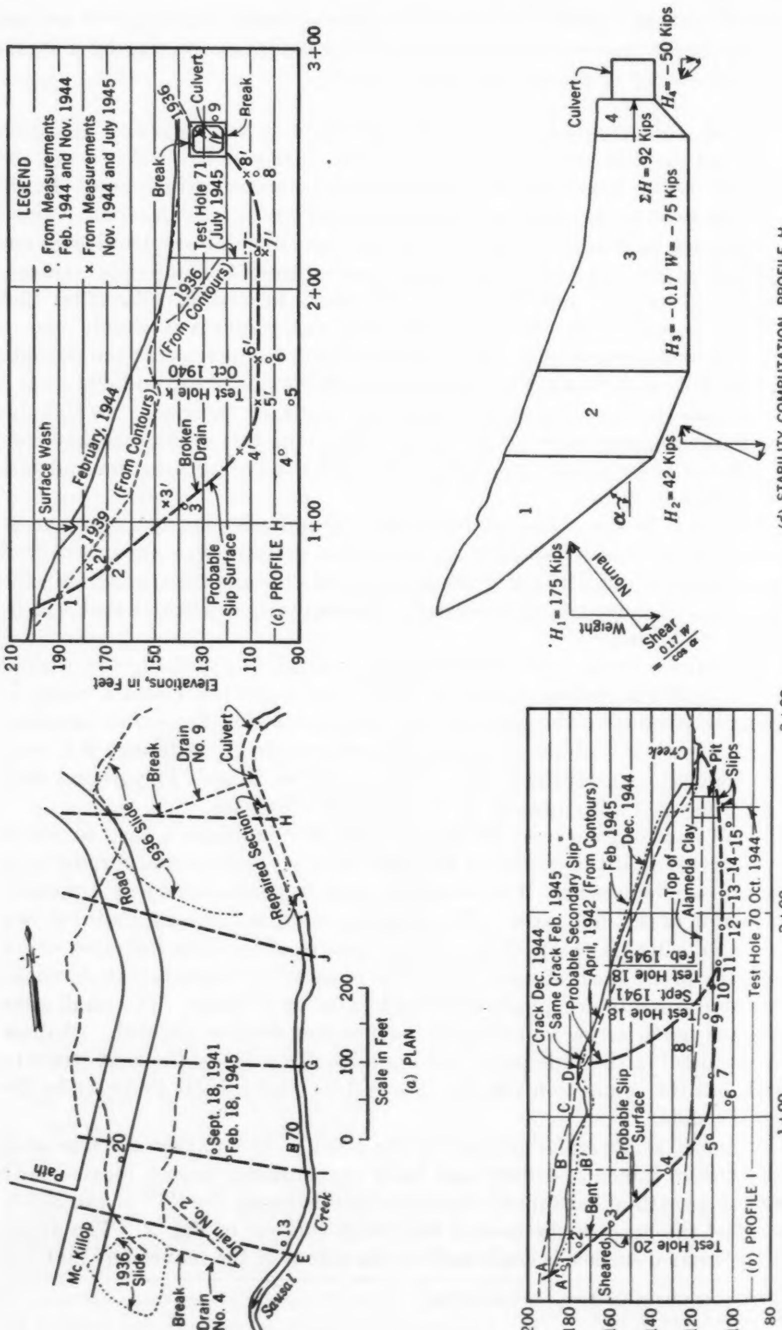

FIG. 16.—MAP OF McKILLOP SLIDE

occurred; but it would be interesting if the author could comment on the location and relative importance of the channels that he found.

A sharp apex forming the head of a slide indicates a narrow, shallow channel feeding into the apex. If the slide broadens considerably in the lower part, it is likely that either the slip surface is deep or there are other feed channels part way down the slide, or both.

Irregular embayments at the head of a slide are almost certain indications of more than one source of water; channels sometimes can be located within a few feet by this method alone. The Simmons Street slide, in Oakland, shown in Fig. 17, is a good example; the three channels shown were strongly suspected

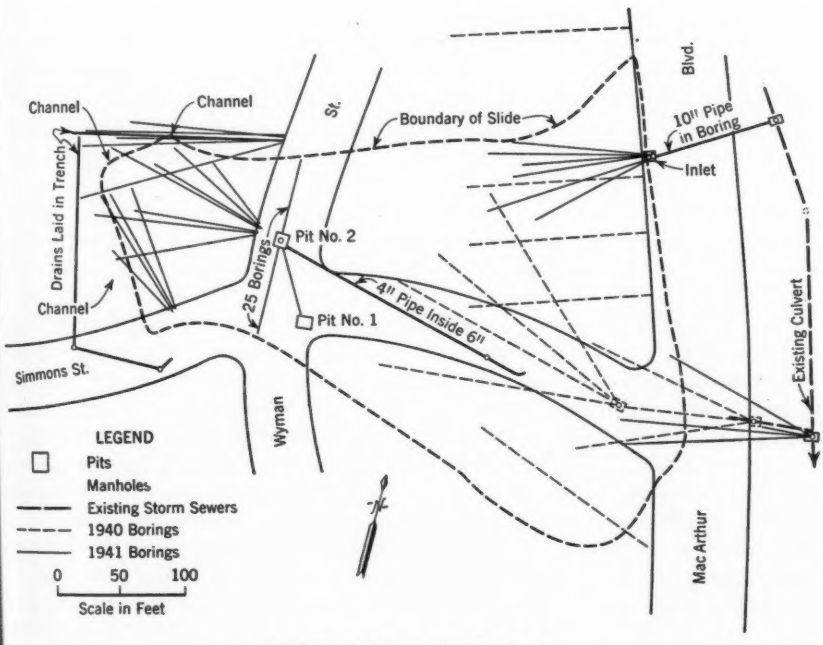

FIG. 17.—SIMMONS STREET SLIDE

before they were located by borings. The apparent embayment on Simmons Street above Wyman Street is an upheaval or secondary toe, and does not indicate a channel.

The terms "shallow" and "deep" are relative, of course, and must be considered with respect to the other dimensions of the slide.

Displaced Volumes.—Under favorable conditions it is frequently possible to compute the shape of the slip surface with a fair degree of accuracy from surface surveys. The method is by no means infallible, but the writer has found it to be of considerable value, and when it fails the error will usually be obvious. The field work consists of running one or more profiles as nearly as possible in the direction of the slide motion, using marked points, and repeating

the measurements after an appreciable movement has occurred. The McKillop slide will be used to illustrate the procedure.

Referring to Fig. 16(*b*) and considering a thickness of unity perpendicular to the paper, the area ABB'A' equals the volume of material which has moved across the vertical line B-4, provided there has been no expansion or consolidation of the slide mass. If the rate of movement has been constant throughout the depth, this area divided by the horizontal component of the movement of point B will give the depth to point 4 on the slip surface. The process is repeated for each point on the survey, the discrepancies evaluated as well as possible, and the probable slip curve sketched in. There are obviously several sources of error in the method, but the operator can rapidly acquire enough skill to make fairly accurate allowances for most of them.

If the slide is fairly uniform at right angles to the plane of the profile and if the line of motion is comparatively straight, any reasonable skew between the profile and the direction of motion should not affect the result, as the cosine of the angle of skew appears in both the area and the horizontal movement. The method does not appear to be of value for a slide of irregular cross section moving on a meandering course.

If the toe of the slide is not being carried away by a stream or other agency, the over-all change in volume of the slide mass can be determined; a reasonable distribution can be made from the evidence presented by open cracks, shattered areas, etc.

An important local error may develop at an abrupt change in the slope of the slip surface if there is rotational motion present. Movement of the survey point will be less than the mean if the slip surface is concave upward, and the computed depth will be too great. This condition is illustrated by points 5, 6, and 15 of Fig. 16(*b*) and 4, 5, 6, and 8 of Fig. 16(*c*). Rotation may or may not be relieved by the development of a vertical crack such as formed at C and moved to D (Fig. 16(*b*)), apparently breaking the slide into two blocks moving on different slopes. A similar condition developed on a smaller scale over points 4 and 5 of profile H during the second measurement period, and may account for the apparent lack of rotational effect on points 4', 5', and 6'. In the somewhat unusual case of a slip surface which is convex upward, it is possible that the computed points would be too high, although expansion due to tension at the surface might introduce another type of error.

If there are two or more major slip surfaces separated by any considerable distance, the motion will not be uniform and neither slip can be located by computation. If the two slips join, the error will be localized, as at points 8, 9, and 10 of profile I. The occurrence of two or more parallel slippages within a narrow zone would not seriously affect the result; the computed position would fall between the actual surfaces and would be well within the zone of slippage. The existence of multiple slippages sufficient to cause appreciable error usually will be indicated by surface disturbances.

After the slip surface has been sketched, the result can be verified by a comparatively few borings. The major part of these can be sunk by ordinary drilling methods, with core samples required for only a few feet in the vicinity of the slip surface. As shown in Fig. 16, the check may be quite close. Point 2

of profile I is known to be too high because of a zone of multiple cracking at the head of the slide; if a similar allowance had been made at point 3, the sketched slip would have been very close to the known shear in test hole 20. The loss of depth of hole 18 is believed to be due to the infiltration of sand, and this well apparently does not reach the main slip surface. Point 15 is obviously too deep because of rotation at the toe; a smooth curve drawn from point 14 to the known toe in the creek passes directly through the upper of two slips found in hole 70. The checks on profile H are not so definite, but are reasonably certain. The broken drain was installed by boring, and may not be exactly on the theoretical grade shown. An abandoned culvert was encountered about 20 ft from the start; a note made at the time suggests the possibility of a downward deflection. Test hole K did not encounter the material in which the slip is known to occur, which merely proves that the slip is below the bottom of the hole. Hole 71 was driven with hand tools. It was nearing the limiting depth for the equipment when it encountered a shattered zone, very similar to one exposed in the pit in hole 70, extending through about 5 ft immediately above the slip surface. This was accepted as sufficient confirmation, and no attempt was made to reach the actual slip surface. If point 9 is disregarded because of a slight loss of volume due to incipient failure of the culvert, the slip drawn from point 8′ to the farthest measured surface movement intersects the culvert at a point which would account for an observed upward displacement of the left part of the failed section. It then continues up the original east bank of the creek, which would be a natural line of weakness.

Some confirmation was available for profile E from test hole 13, the behavior of drains 2 and 4, and from an exposure of the slip surface by creek cutting. Profiles G and J showed characteristics similar to the other profiles, and were accepted without confirmation.

The writer has had very little success in trying to fit circular arcs to actual slides, and therefore is not satisfied with the conventional methods of making stability computations. Most slip surfaces can be represented satisfactorily by a comparatively few straight lines and the section divided into prisms with plane bases. Then the only assumption necessary is the interfacial pressure between the prisms. The frequency of vertical shear displacements at important changes of slope of the slip surface indicates that the error in assuming this pressure to be horizontal would be comparatively small, particularly after vertical cracking has occurred. A force polygon can be drawn then for each prism, and the horizontal forces summed, as shown in Fig. 16(*d*). This procedure is not strictly correct in that the difference of elevation of the lines of interfacial pressure has been neglected, but the simplification is believed justified.

The greater part of the slip surface of the McKillop slide lies in a very firm silty clay locally known as the Alameda clay, which underlies the hill just above creek level. This clay has a shearing strength of well over 1,000 lb per sq ft in its undisturbed state, but becomes very plastic when remolded. Shear tests, by both direct shear and unconfined compression on completely remolded material extruded from the slip surface at the toe of the slide, indicated a shearing strength of 0.17 times the reconsolidation pressure. The material overlying

the Alameda clay at first appearance seems to be entirely different in character, but an examination shows that its clay content gives it similar shear properties. The value of 0.17 was used, therefore, for the entire slip surface. (It should be noted that the writer carefully avoids any use of the expression—

$$s = c + p \tan \phi \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

—which he regards as an unfortunate mathematical coincidence. In Eq. 8, s is the shearing strength; c is cohesion; p is the normal pressure; and ϕ is the angle of internal friction. Expressed in this nomenclature the test results would have been $c = 0$ and $\tan \phi = 0.17$.) The unit shearing strength on the slip surface is 0.17 times the vertical pressure per unit area; the total shear for the prism is 0.17 times the total weight of the prism divided by the cosine of the angle of inclination of the slip surface. If this analysis is correct, the conventional conception of cohesion plus friction introduces an appreciable error on the side of safety, which may help to explain why the thin factors of safety in current use have not more often resulted in disaster. The writer was led to his conception of shear by the close agreement between the results of direct shear and unconfined compression tests. It is difficult to reconcile the unconfined compression test with a zero cohesion and a coefficient of friction; whereas a simple shearing strength determined by consolidation pressure (or consolidation history of an undisturbed sample) would seem to fit both tests.

A valuable check on the computation is afforded by the behavior of the culvert. Profile H, Fig. 16, was established in the direction of previously recorded movement, and is somewhat skewed with respect to the culvert. On the other hand it is quite probable that the thrust on the culvert is increased by an arch action from the mass of the slide, indicated on the ground by a series of cracks which disappear about where they cross the line of the profile. The profile crosses the culvert near the upper limit of the failed area, and the computed slide thrust of 92,000 lb per ft compares with a value of 93,500 lb per ft for the failure strength of the culvert. This seems to be well within the accuracy of any current method of determining the shearing strength of soil.

Horizontal Borings.—In fields of construction that have not become standardized the engineer must of necessity adapt his methods to suit available equipment and the skill and experience of local contractors. Whereas the author has developed a technique of wells connected by short tunnels, the writer has had considerable success with a machine for making horizontal borings. The essential part of the machine is a reversible air motor with a hollow crankshaft which delivers water to a cutting bit through a string of hollow rods. When working in clayey material, it is capable of surprising accuracy, but is almost impossible to control in gravel. Progress is rapid, and in 1940 a 4-in. perforated pipe, in place, cost about $1.00 per ft for drains averaging 100 ft long. The cost increases somewhat for longer drains, and increases rather rapidly with the diameter of the pipe installed.

McKillop Slide.—The history of the McKillop slide illustrates several interesting processes of slide development. A competent observer reported topographic indications of considerable prehistoric movement of the northern part of the slide, between the culvert and the street. Cross sections taken prior

to the construction of the street show a substantial gully at this point, which was filled by street grading. By 1935 the fill had been extended, presumably by loose dumping, far enough to permit the construction of three houses on the east side of the street. This overloading of the slope, possibly aggravated by some creek cutting, was sufficient to cause collapse during the wet winter of 1935–1936. Later in 1936, the culvert shown in Fig. 16(a) was constructed in firm ground along the east bank of the creek, and fill was placed to the height of the east bank.

At about this same time a small slide occurred in another filled gully on the east side of McKillop Road just south of the present limit of the main slide. This appears to be a superficial movement more or less independent of the main slide.

The loss of support due to the northern 1936 slide apparently caused a local concentration of stress sufficient to rupture the Alameda clay immediately south of the 1936 slide. Slow cracking of the houses on the west side of the street, progressing from north to south, indicated that this concentration of stress and consequent rupture traveled southward until it reached the small southern slide. These movements did not reach alarming proportions until the spring of 1940. During the summer of 1940 several drains were installed, and both of the 1936 slides were refilled in an attempt to buttress the slide mass. The culvert was located on developed property, and, therefore, it was not possible to place additional fill in that area. The corrective work was not successful; movement has continued to the present time, with the destruction or removal of all improvements in the area, including the partial failure of the culvert.

Drainage work included several trench drains in McKillop Road and a number of borings made from near creek level. A considerable volume of water was removed; attempts at drainage were abandoned when it was definitely determined that the major part of the slip surface was below creek level. The construction of a counterweight fill at the toe would require the condemnation of developed property on the east side of the creek, and does not appear economically justified. The culvert was considerably larger than required to carry the creek, and has been repaired by installing a steel plate liner backed with asphaltic concrete. There are indications that the slide is now (1946) heaving on the west side of the culvert and forming a new toe which may ride over the culvert instead of thrusting against it. In that case the present flexible culvert will probably not deform enough to cause failure.

Simmons Street Slide.—The Simmons Street slide (Fig. 17) developed in the winter of 1939–1940, and for two winters threatened to move across MacArthur Boulevard and block the main branch of U. S. route No. 50. There is considerable evidence that the slide was a renewal of an ancient movement of practically the same mass. A crack along the south edge of the slide and an upheaval near the southwest corner of Simmons Street and MacArthur Boulevard seemed to be duplicating a gully and knoll described by persons who had been familiar with the area before it was graded. If this former movement did occur it must have been very old, as all traces of a scarp at the head of the slide had disappeared.

During the summer of 1940 a number of drainage borings were made in the lower part of the slide, as shown by the dashed lines in Fig. 17. This work did not prove effective, and movement was resumed in January, 1941. This second movement was sufficient to outline the head of the slide definitely and to permit the location of the water channels shown, which were intercepted by a combination of borings and drains laid by ordinary trenching methods.

Two test pits were dug on Wyman Street and a line of test holes put down to establish the cross section of the slide. The deeper of the two pits was then drained by boring from a point on Simmons Street about 175 ft away. The boring was cased with a 4-in. pipe inside of a 6-in. pipe to provide a slip joint in case of further movement. The boring reached the center of the pit at a point about $\frac{1}{2}$ ft above the planned elevation. It was planned to construct a shaft and a tunnel drain across the slide, similar to the one described by the author at Burnham Street and 24th Street, but the one bid received was considered excessive. As a substitute twenty-five borings were made along the line of the proposed drain, working from a series of three stagings in the pit. In the critical places the borings were about 1 ft apart vertically, although staggered somewhat in plan. The cost of the work was approximately one fourth of the amount of the rejected bid, and the cutoff seems to have been complete. Some difficulty was encountered in sluicing the wash through the outlet boring. It would have been better to have deferred the installation of the inside 4-in. pipe until after the work in the pit had been completed had it been known that the cutoff would not be constructed as planned.

Additional test holes along the toe of the slide showed that the 1940 borings were not deep enough, and the outlet of the group in Simmons Street had been destroyed by the 1941 movement. A pit was opened over an existing culvert east of MacArthur Boulevard opposite Simmons Street and a new group of borings was made at a lower level. As the water in the north side of the slide was beyond boring range from the culvert it was necessary to bore from a pit over the culvert to another at the west curb line of MacArthur Boulevard and make the drainage borings from the latter. Since the drain connecting the two pits was required to carry surface water from the gutter, it was cased with a 10-in. pipe. Except as indicated, all other borings were cased with 4-in. perforated pipe.

Transit lines were established along all streets in the slide area in February, 1942, and remeasured at intervals until September, 1945, with no measurable movements detected other than those caused by local swelling of the clay soil.

This slide was stabilized before the method of displaced volumes was developed. It is somewhat irregular both in cross section and direction of movement, and exhibited a definite spreading action as it moved downhill. As both irregularity and lateral expansion introduce serious errors the method probably would not have been successful.

Barrows-Holman Slide.—This slide, which is shown in Fig. 18, is typical of several which have occurred in a rather limited area of Oakland. The tract was developed by the owners between 1922 and 1924. Apparently the fill on the north side of Holman Road was placed with very little compaction and no subdrainage. Minor repairs to sidewalks and drives on Barrows Road had

been required for years, and considerable settlement had occurred along the north side of Holman Road; but major slide action was not recognized until the early summer of 1942. At that time the toe upheaval along Barrows Road was fairly well defined although no serious damage had resulted, and the head of the slide on Holman Road was partly outlined by a series of sidewalk and driveway cracks not wider than 1 in. The adjacent settlement delayed, somewhat, the determination of the exact limits of the slide but the shape was well enough defined to point to a small area on Holman Road where the cracks extended into the street. As the original cross sections showed a fill of about 7 ft on the north side of the street it was obvious that a spring or seep had been covered and that the percolation of the dammed water through the years was responsible for the slide. A series of test holes along the street located the channel almost

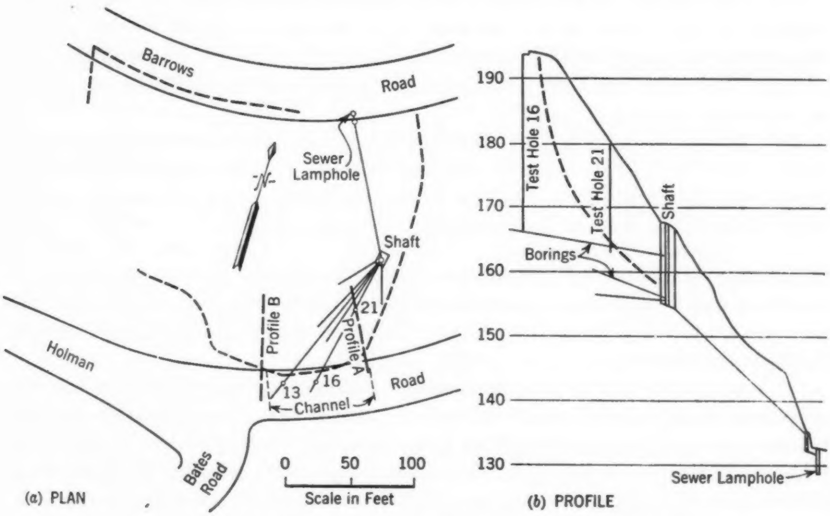

FIG. 18.—PLAN AND PROFILE, BARROWS-HOLMAN SLIDE, OAKLAND, CALIF.

exactly as anticipated, even to its width. Several 8-in. wells were then drilled in the channel and plans were made for drainage. The greatest known depth of the channel was 28 ft, which would have made a trench drain long and expensive. Two profiles were established in the upper part of the slide, and, although the recorded movements were only a few hundredths of a foot, and therefore scarcely larger than the inaccuracies of the survey, the result seemed reasonable. Test hole 21 (see Fig. 18) was then put down and the slip surface was found within a foot of the computed elevation.

As soon as financing arrangements could be completed, a shaft was excavated near one side of the slide to an elevation well below the slip, and drained by boring from Barrows Road. Borings were then made from the shaft to drain the channel and pick up such additional water as could be intercepted by a few random borings. The boring to hole 16, a distance of slightly more than 100 ft, actually intersected the well, and the drain pipe was clearly visible in the bottom

of the 8-in. well. The boring to hole 13 (about 120 ft) was never seen from the surface, but welding fumes were quite strong at the well while the casing was being installed in the boring.

It was realized that the widening toward the west of the lower part of the slide indicated the possibility of a deeper water feed in this vicinity; but no such channel could be found readily. As it was necessary to keep costs as low as possible, it was decided to defer work in this vicinity until it became necessary; and the subsequent stability of the slide seems to have justified this decision. The slide was arrested after a movement of probably less than 6 in., and the damage caused was minor.

Conclusion.—In conclusion the writer wishes to emphasize the following points:

(1) The problem of landslides is one involving both geology and engineering, whether the investigation is conducted by a geologist with an appreciation of engineering problems or an engineer who has acquired a working knowledge of geology. In most cases the only corrective measure required is the removal of underground water.

(2) The science of soil mechanics has not yet reached the state of standardization where its methods can be accepted without careful analysis and review. The combined work of many men is yet required to bring it to a state of reliability comparable with structural design or hydraulics.

ALFRED V. BOWHAY,[16] Assoc. M. ASCE.[16a]—The subject of landslide correction is one that merits a complete review such as Mr. Forbes presents. It is of particular interest to those engaged in municipal engineering as well as in highway engineering.

The municipal engineer is vitally concerned because varied types of municipal improvements involve the removal of support through excavation and the deposit of materials in constructing embankments, both of which (through the action of moisture) are potential slide situations. With a better understanding of the action of the forces of nature, effective measures can be taken to prevent, or to minimize, the destruction of valuable public and private works. It is not within the scope of the civil engineer's practice in municipal engineering to apply any corrective measures other than ordinary drainage. The engineer is competent to study borings and strata so that drains may be placed at the proper locations and depths, thus insuring drainage of excess water. He is also competent to apply surface measures such as consolidation by rolling and the application of asphalt or cement coatings to divert surface drainage to the regular storm drainage system. In this manner the sliding material may be relieved of seasonal rainfall, and will have an opportunity to drain and dry out, so that it may again reach a state of stability.

Beyond this practice the civil engineer cannot reasonably go—especially in municipalities where only a few slides occur from year to year. In San Francisco, Calif., there have been some eight or ten major slides in a period of thirty years. This record is too limited to establish a standard norm of behavior.

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[16a] Received August 12, 1946.

To obtain adequate knowledge of the causes and remedies, it is necessary to procure the services of a competent engineering geologist, experienced in the particular procedures peculiar to landslide correction. Such a geologist should have experience on from fifty to one hundred landslides.

There is a technical aspect of the procedures of the geologist that Mr. Forbes mentions—namely, the tests, in the Crolona Heights slide, of plastic limit, liquid limit, and moisture equivalent. These tests indicate the type of soil or clay, the amount of surplus moisture, and, more particularly, the moisture retention qualities. Because of these factors, some ground loses its moisture by drainage so gradually that three or four years are required for it to dry out to a condition of stability. This slow progress toward stability, and the continued sliding after drains are installed, discourages engineers from trying to stop slides. However, a geologist, with wide experience, through knowledge of comparative conditions, can advise the correct type, number, and location of drains to suit the conditions that these tests indicate. Thus, dewatering or drying out can be reduced to the minimum time economically justified.

The correction of slides involves procedures of great interest to engineers because of the complexity of the problems involved. Often the land is worthless before the slide. There is little value in a cut slope, for example; and, after a slide occurs, the value of the land involved is reduced to a negative sum in most instances. Highway or street frontage must be cleared, and it is essential to repair the area to prevent a recurrence. The question naturally arises as to whether such work will warrant an expenditure greater than that necessary to clear up the debris. In this respect, "the wish is father to the thought," and the general thought is "perhaps it will not slide again." In nearly every instance, cities proceed on this basis. They clean up the debris, put in a minimum of subdrains, and repave or replace the improvement if possible.

The nominal cost of a geologist's services is well worth the public confidence it inspires. Public administrators cannot justify the expenditures frequently called for, except with the best advice obtainable. Even private individuals, homeowners, or those in charge of institutions threatened by slides cannot readily question the technical advice of an engineering geologist, as they quite often question proposals by engineers, made without the benefit of geological study and advice.

The term "maintaining a slide" appears to be on the facetious side, but to maintain a slide has been found to be a necessary and often valuable procedure. In the Laidley Street slide, one of the first major correction jobs of the City of San Francisco, after the drains were installed, the surface was smoothed and rolled with a heavy tractor (used because of a grade of about 20%) and then given a coating of about 0.5 gal of heavy road oil per square yard. This surfacing acted as an umbrella, and it shed the water to the foot of the slide where it was conveyed to the storm sewer. Naturally in a few years the surface deteriorated, and insufficiently compacted pockets developed; but, by regular annual inspections and study, the conditions were noted and remedied. It was even observed that there were points where additional drains should be installed.

A part of the maintenance work is to keep a discharge record of the drainage system. This record is important in connection with observations of rainfall.

Reference stakes that show the progressive horizontal and vertical movement must also be maintained and observed. The study of these data gives valuable aid in improving design by such measures as added drains and improved surfacing, as well as in showing the degree of improvement produced by the corrective work.

It seems advisable to direct attention to the possibility of prescribing general treatment, which Mr. Forbes indicates cannot be done (see heading, "Correction Should Be Based Upon a Thorough Investigation"). Since 1926 the City of San Francisco has used the general treatment of drainage by the construction of ditches with the familiar perforated metal pipe. Later on this drainage was accomplished by chimneys. These chimneys or vertical drains are usually extended downward to a nearly horizontal drain, where the usual metal pipe and rock backfill is installed. This type of drainage was installed in the Parker Avenue slide in San Francisco as reported by Mr. Forbes. The design of the Parker Avenue slide drainage system was prepared by, and under the immediate direction of, the writer. This system was the first of its type, subsequent to which the 3.5-ft by 6-ft drain tunnel was replaced by a very small tunnel (minimum 15 in. by 15 in.) connecting the bottom of the shaft. The latter work was done on the O'Shaughnessy Boulevard slide (see Table 1 and supporting text) also in San Francisco. A perforated metal drain with crushed rock backfill is usually installed in this small temporary tunnel. Lately the shafts have been increased in size to 4 ft by 5 ft, so that they could be timbered and still admit a small clamshell bucket. The cost of this type of shaft is less than that of the 36-in. circular shaft, the excavation for which is all handwork and requires a metal 36-in. shell for lagging. The 36-in. shell sometimes cannot be recovered during the placing of the center drain and rock backfill, generally constructed to within a few feet of the surface.

A great advantage of this general procedure is that a suitable contract can be drawn so that competitive bids may be taken. Quantities can be determined satisfactorily in advance, necessitating a minimum of change in the actual work. This change is always well within the 10% allowance for extras, the limit required by city ordinance. Of course, this contract preparation must be preceded by borings (by contract) and sample studies and tests. From such data, profiles and sections of the strata can be made for study so that, as Mr. Forbes states, corrective works can be designed to meet these determined elements of the problem.

The nine classifications presented in the "Synopsis" are interesting, although they appear to overlap slightly. No doubt there is ample geological reason for differentiation. However, of particular interest is "(1) a shear slide in which hydrostatic uplift on an unbalanced slope resulted in mass movement of the unconsolidated slope material." This class of slide is further subdivided under the heading, "Landslides"): "Shear slides are frequently associated with the unbalanced slopes that remain after material has been removed from natural slopes." Although Mr. Forbes does not apply this type to the Parker Avenue slide, there were many persons who did. The idea that the particular slope in question was unbalanced, or that there was an unbalanced condition, seems to vitiate the theory of safe slopes, which is still popular in the practice of highway

engineering. Under the theory of safe slopes, certain materials will stand at a slope of 1 on 1, others on a slope of 1 on 1.5, and the most unstable excavated materials on a slope of 1 on 2. On Parker Avenue these slopes stood at a slope of 1 on 0.5 for nearly fifteen years. There was some seepage which was provided for, showing a wet ground condition. However, it could be concluded that the slope was safe, particularly on the Lone Mountain side of Parker Avenue, which had a concrete and brick pavement, 35 ft wide. Across Parker Avenue, in a sidehill situation, was the low slope of the cut which was removed or "daylighted" during the subdivision work. This construction was claimed to have unbalanced the material across the street and up the hill. If this were true, then the material could not have been classified as safe for a slope of 1 on 0.5 fifteen years previously. Actually, unbalancing was not a primary factor in this situation, but it was a minor factor in resisting hydrostatic uplift.

In the Parker Avenue slide, water was the primary factor, as the millions of gallons drained from the slide annually will attest (Table 2). The material was stable but highly absorptive, and had a high retention value. There was also the familiar slipping plane of plastic material, which had no resistance to sliding when wet. The great mass of saturated material had an unusually heavy weight, the resultant of which was parallel to the slope of the sliding plane. The weight of the counteracting dry material, which was similar to that of a dam, held this resultant force of the wet material stable. As the saturation extended down the slope, and the slipping plane became further lubricated, the resistance of the dry material to sliding became so low that, like a poorly designed dam or retaining wall, it was thrust aside. The plan of the slide, as well as the photographs, shows this lateral movement. The apparent uplift resulted because the slipping plane turned upward and the material was also thrust upward along this plane. The plan also shows the trend to the northwest of the gulley which formed the bed of the slipping plane.

Unbalancing, therefore, entered into this slide problem only in the sense that the weight of counteracting material, acting as a dam, may have been reduced. The heavy weight of the wet material, and the moisture-lubricated slipping plane of plastic character were the prime causes of movement, which could have occurred whether the excavated material was or was not removed. Slide problems will be more clearly understood if this distinction is kept in mind. Although engineers may speak of slopes in balance and discuss them in such a manner, actually in engineering problems of this kind they are referring primarily to the forces that are in balance.

One of the more important factors in the solution of slide problems is the force that resists sliding. As usual, this force depends on the weight of the resisting material and the coefficient of friction, which is naturally very low when the sliding surface lies along or within a seam of wet plastic material.

Landslides not only damage highways, streets, and public works, they also damage homes. Some fortunate homeowners can remove their houses to a more stable location. The remaining foundation on private property rapidly deteriorates and becomes a public nuisance and eyesore, a kind of testimonial of the inability of the community to cope with the situation. The less fortunate homeowners, whose houses are so badly wracked and damaged by the slide

action, generally appeal to the local governing agency for help or assistance. This help the local agencies, supervisors, or councilmen feel obligated to give. The repairs or replacements are too often makeshift in character because temporary measures are popular in slide correction. Thus, most of the slide areas become not only vacant land but also eyesores. This is a poor testimonial to the engineer and his ability to sell his measures toward a more complete remedy.

More education and enlightenment are needed on this subject. Engineers and engineering geologists must collaborate to convince their communities that these eyesores can be corrected, and that the cost of the remedies is not out of proportion to the benefits. Then one may look forward to less disfiguring areas in the rapidly growing modern cities and along the modern highways of the United States.

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DISCUSSIONS

EXPRESS HIGHWAY PLANNING IN METROPOLITAN AREAS

Discussion

BY JACOB FELD, HAROLD M. LEWIS, THEODORE T. MCCROSKY, SPENCER A. SNOOK, LAWRENCE S. WATERBURY, BERNARD L. WEINER, AND GEORGE H. HERROLD

JACOB FELD,[17] M. ASCE.[17a]—By assembling and organizing the pertinent data as well as the differences of opinion covering all phases of the problems involved in the planning of express highways through congested areas, the author has prepared a valuable contribution. There is considerable published literature on individual phases of the problem and even more on the designs prepared for special locations. However, a paper of this type which looks upon the problem as a whole, without any special reference to a definite locality, provides a framework for the use of the designer working on any phase of this problem.

Proper planning and execution of express highways will reduce congested traffic and expedite the flow of vehicles through urban areas. However, it would be uneconomical to attempt a complete elimination of congestion in large cities. As a matter of fact, the very basis of city existence is congestion and an efficient city cannot exist without large concentrations of vehicles. The popular expression for an inefficient city, where concentration has been reduced to the point of elimination of congestion, is "an overgrown village." People expect congestion when they enter cities and to some extent would be disappointed if they found none—for instance, the tendency of crowds to congregate on certain streets in every city, most of the sight-seers having no particular business in those streets.

An attempt to design through arteries for every possible concentration of traffic (such as that occurring for a few hours each week end during vacation months in a comparatively few roads leading to every large city) would be a tremendous waste of the facilities for practically 99% of the time. Fig. 7

NOTE.—This paper by Joseph Barnett was published in March, 1946, *Proceedings*. Discussion on this paper has appeared as follows: May, 1946, by Harry W. Lochner, and Fred Lavis; and September, 1946, by Homer M. Hadley, Donald M. Baker, W. J. Van London, Merrill D. Knight, Jr., and R. H. Baldock.

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[17a] Received July 2, 1946.

shows the traffic congestion along the West Side Highway in Manhattan (New York, N. Y.) during the afternoon of Navy Day in 1945. Practically the same degree of congestion exists for about two hours on this highway every Sunday afternoon when the weather is favorable. Possibly a doubling of the capacity might eliminate this congestion for these short periods but it would not be economical to provide such additional facilities.

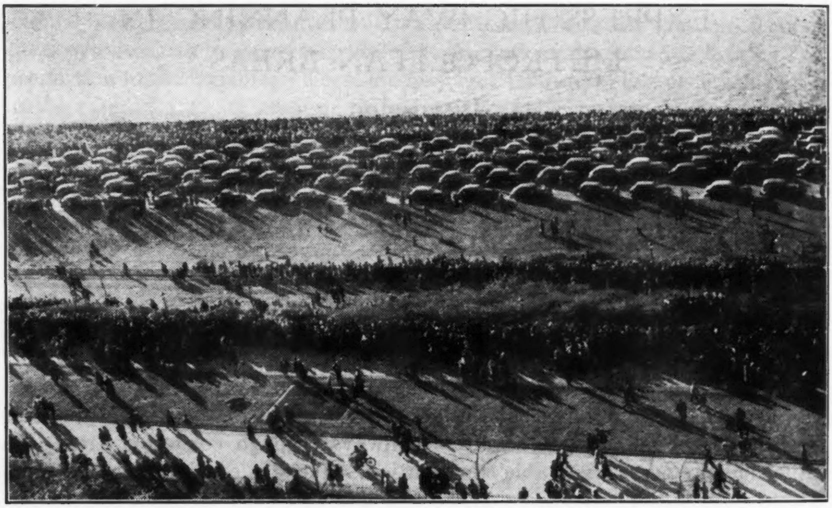

FIG. 7

One method to reduce incidental peak concentrations would be to revise the method of marking through routes. From personal experience, it has also seemed to the writer that it is a mistake to mark certain streets going through a city with a definite route number—especially where a number of routes converge into a city center and then diverge on the opposite side of the center. It would help traffic if the general direction routing was provided for the motorist rather than a definite route number. For instance, a number of streets leading from the outskirts on the west side of the city could be marked east traffic and a very simple sign in the form of an "E" with an arrow could be easily identified. Traffic would then split through a number of streets rather than concentrate on a single route. Such marking would be advisable even where a through artery is provided to take the overflow during the congested periods.

The writer has found that much better time and less nerve-wracking driving results from by-passing marked routes. For instance, in the City of Waterbury (Conn.), all routes intersect at the center of the city. The provision of a "throughway" for all traffic, converging on the center from practically eight different directions would be uneconomical. A circular or square connecting highway through the outskirts of the city would help the city congestion very little (as the author states), because most of the traffic is going to different parts of the city. However, provision of a throughway going east and west and one going north and south, of minor traffic capacity, with roughly parallel

streets marked with direction would solve the problem. This solution is similar to a design of a main pipe line with parallel by-passes for surplus or surge flow. A similar analogy can be found in multiple electrical connections in a grid system.

Urban traffic concentrations have been studied by traffic divisions of the New York City Police Department for many years. Much information can be obtained on peculiarities of traffic congestion from those sources. For instance, in 1936 the writer was organizing construction of a part of the Sixth Avenue Subway in Manhattan, and among other things desired to find which days would have the least traffic concentration. The street had to be closed for short periods at a time and to avoid too much interference with the local business people it was thought advisable to plan the work so as to close the street on the lightest traffic days. Of course, streets are very often closed for repaving and for other construction, and traffic somehow finds its way around and takes care of itself. On the other hand, the experience of the police indicated that in retail areas like that part of Sixth Avenue, vehicular traffic was heaviest on Fridays.

The question was important enough to warrant the expenditure of some time, and traffic counts were made from April to December, 1936. Actual count of the number of vehicles passing the northerly intersection of 41st Street and the northerly intersection of 45th Street, on Sixth Avenue, were made for a period of 5 min each hour from 8 a.m. to 6 p.m. on six successive days (except Sunday) starting on the fifteenth of each month. Fig. 8 shows the variation of traffic averages for various months. At the time of the count, the

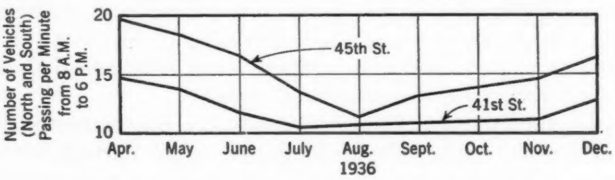

FIG. 8.—TRAFFIC COUNT ON SIXTH AVENUE, MANHATTAN

roadway width of Sixth Avenue was 60 ft with an elevated structure in the middle of the street. The clear distance between elevated columns was 22 ft with unused trolley tracks in this part. Each side road was 17 ft in clear width. Theoretically, parking was prohibited, but actually delivery trucks and some private cars were almost always at each curb at each block. The results of the traffic count were used to schedule the main closure of the streets for the summer months when traffic concentration was least and also to avoid closing the streets on Fridays as much as possible, even though the increase in traffic counts for Fridays was not very much above the other days of the week.

This example is cited to emphasize the difference in traffic concentration when commercial traffic is being considered rather than pleasure vehicles. The fact that truck concentration is not at a peak during week ends is often forgotten because of individual experiences of being tied up in vehicular traffic on Sundays.

There are two schools of thought on the location of interchanges between an express highway and intersecting main streets. One group feels that the interchanges should lead directly into the main street, to avoid addition of traffic in adjacent and usual residential areas. The opposing thought is to connect the interchange into subsidiary streets to allow the traffic leading or entering the expressway to filter through supplementary streets and thereby reduce congestion on the main streets, which usually already carry considerable traffic. The decision is difficult to make, especially where the main street is bounded on each side by residential land use and often by heavily concentrated populations. The meandering of traffic entering or leaving expressway streets serving apartment houses adds seriously to the traffic accident toll. From the safety point of view, it might be better to force all traffic on to business streets even though some congestion results.

The author must be complimented for the very clear manner in which he has analyzed this entire problem and for the open-minded discussion of different solutions for each phase. The design and construction of express highways is becoming a considerable part of the engineering and construction fields. The crystallization of thought and opinion at this time is therefore most useful.

HAROLD M. LEWIS,[18] M. ASCE.[18a]—The need of coordinating main arterial highways with general city plans, and of planning suitable off-street terminals for passenger cars and trucks in urban centers, is clearly demonstrated by this important paper. The writer has found that many highway engineers are interested in city planning, but more of them should be; in turn, city planners should give more attention to the economic justification of the highways they propose. A master plan for major urban thoroughfares should do more than create a pattern of loops and radials; these latter should be designed to meet local demands in location, character, and capacity.

The initial development of expressways in the United States started with parkways designed to carry heavy passenger-car traffic through suburban areas. In a few cases such special routes may extend far enough to become interurban; but, essentially, the parkway has remained a feature of metropolitan development. Postwar highway plans, such as those in New York and California, will place more emphasis on interurban expressways for all types of traffic—passenger cars, buses, and trucks.

Most expressway problems still requiring engineering research arise in urban centers, where, as Mr. Barnett states, the proper location and design of "ons" and "offs" are important. These can be closer together on the edges of the business district than in suburban areas where their location is fixed primarily by that of major intersecting highways along the express route. An excellent example of the downtown loop, typical in a pattern for large cities, will be provided in Los Angeles, Calif., under its expressway program—partly completed, with other sections under construction, but mostly still in the planning stage.

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[18] Cons. Engr., New York, N. Y.

[18a] Received July 12, 1946.

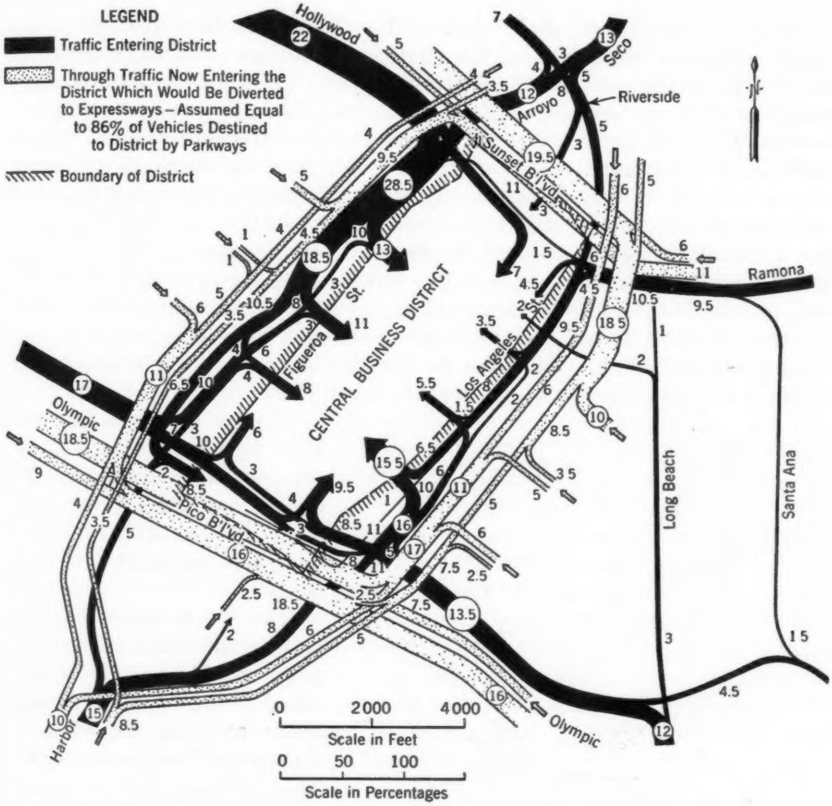

Fig. 9.—Estimated Distribution of Passenger Vehicles Entering and By-Passing the Central Business District Shown as Percentages of Total Entering by Parkways with Present Distribution of Population

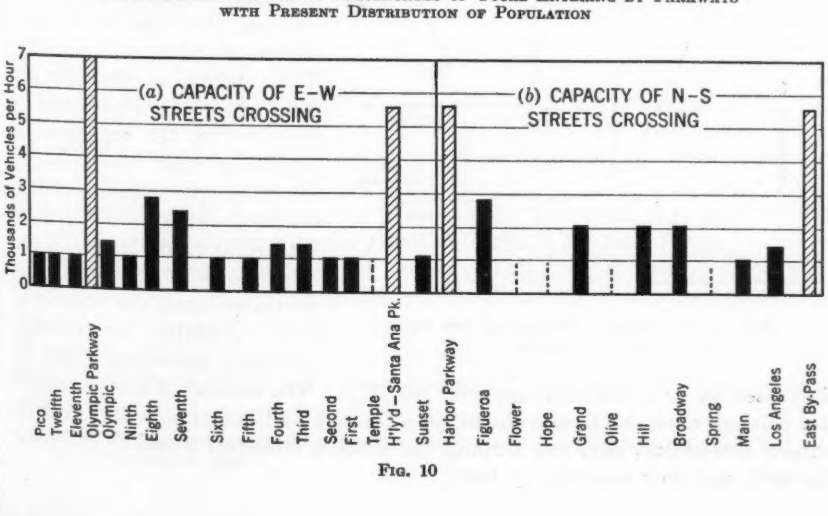

Fig. 10

At a "transportation clinic" held at Los Angeles in December, 1945, the writer presented an analysis of the relative proportions of through (by-pass) traffic that might use these loop expressways as compared with traffic that would use them for entering (or leaving) the central business district. It was recommended that entrances and exits be provided at twelve points. Fig. 9 indicates that the heaviest by-pass traffic would be along the north and south sides of the loop and that the easterly section, known as the "East Bypass," would carry more through traffic than the westerly section, a part of the "Harbor Parkway." Of the through traffic now entering the district, the part destined to be diverted to expressways was assumed equal to 86% of all the vehicles entering the district from parkways. This assumption and the distribution of vehicles among the quadrants of the area were based on origin and cordon counts made in 1939. For the location of "on and off" points it was assumed that equal traffic would move in reverse directions.

FIG. 11.—EXPRESSWAY PROGRAM OF THE CITY OF LOS ANGELES

It was proposed that the East Bypass be used for all types of traffic—passenger cars, buses, and trucks—but that other sections of the loop be limited to passenger cars and buses.

Fig. 10 indicates the increase that the loop expressways would provide in the capacity for traffic moving across or along the edges of the central business district. The solid bars show the hourly load that could be carried over existing, continuous streets crossing this district at their points of minimum capacity. The hatched bars show the additional capacity that would be provided by the expressways. Assuming that the Olympic Parkway (the southern section of the loop) were to be built with eight traffic lanes and the other elements of the loop had six traffic lanes, the number of lanes available for through east-west movement would be increased by 39% and their capacity by 63%. The number of lanes available for through north-south movement (now limited by the fact that four existing streets extend only part way through the district, however) would be increased by 60% and their capacity by 100%.

In addition to about ten miles of existing routes, the complete system of expressways for Los Angeles city and county involves 358 route miles and a total cost of $582,000,000; of this, about 220 miles costing $400,000,000 will be within the city. Of the proposed system, 287 miles have been scheduled under a ten-year program; the remainder are unscheduled. The total of existing and proposed routes, differentiating between those scheduled and unscheduled, is shown in Fig. 11.

The panel of consultants taking part in the "clinic" agreed that about 54 miles of the expressway system should have set aside, as a part of the cross section of the routes, separate rights of way for mass transportation vehicles. The writer proposed that a little more than one third of the 297 miles existing and scheduled should have roadways available to passenger cars, trucks, and buses, and that an approximately equal mileage should be reserved for passenger vehicles. Of the remainder, both private passenger cars and a limited number of express buses might be permitted to use the roadways.

A check on Mr. Barnett's statement that the "By-passable traffic is about 20% in cities with populations of from 10,000 to 300,000" is provided by a survey made on a Saturday afternoon in April, 1946, at Glens Falls, N. Y. (1940 population 18,720). For a 2-hour period license numbers were recorded of all vehicles entering and leaving by the four main highways. Within this period it was found that 19.8% of the vehicles checked crossed the city and left again in a different observed direction. About half of this through traffic passed through without stopping and half stopped for a business or other errand. On a typical summer week end, when Glens Falls is traversed by through traffic to or from the Adirondacks, it is expected that the proportion of through traffic would be considerably higher and a second and longer count was made on July thirteenth to determine what the result would be.

THEODORE T. McCROSKY,[19] M. ASCE.[19a]—This paper contains a very able and complete statement of the need for express arteries through large cities and metropolitan regions, rather than up to their boundaries or by-passing them. The United States Public Roads Administration is empowered to make grants-in-aid for urban arteries, thus enabling many more states and cities to undertake this type of construction, with its heavy but necessary costs, than was possible when federal aid was restricted to highways outside city limits. It will be recalled, as a direct parallel, that many states did not traditionally contribute to the cost of highways within incorporated municipal jurisdictions. The change of state and federal financial policy now makes it possible to construct the highest type of traffic routes progressively outward from the urban center. Previously they generally had to be built inward from open country, with the center itself (where traffic is most congested) too often left with no arteries of modern free-flowing design. As the author has stated, traffic load diminishes rapidly with distance from principal cities, so that the policy of building outward rather than inward is manifestly correct from a technical standpoint.

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[19] Executive Director, Greater Boston Development Committee, Inc., Boston, Mass.

[19a] Received July 22, 1946.

The proposed central artery through downtown Boston, Mass., is a case in point. It will receive the heavy streams of entering surface traffic, converging from the northwest and south toward its extremities, and lead them to, or through, the heart of the city. This project is sponsored by the city planning board, and the 1946 session of the state legislature authorized the Post-War Highway Commission to draw upon the highway fund (gasoline tax revenue) for preliminary surveys and engineering. It is planned to provide express connections with the Sumner vehicular tunnel leading to East Boston, and with the twin tube of this tunnel when constructed. Extension of the Central Artery outward beyond its currently planned termini might be a matter for future consideration, but is by no means as urgent as the construction of the downtown section.

The proposed Boston Central Artery also furnishes an excellent example of selecting right of way through a blighted "fringe area," as defined by the author, which is only a few blocks from principal central destinations.

The writer wishes to stress the importance of careful traffic counts and estimates for peak 15-min and 30-min periods, as a necessary basis for the sound determination of the correct number of vehicle lanes to provide for express highways. General, 24-hr counts are important in estimating total flow, particularly for toll facilities; but they do not answer the question of critical traffic loads that may result in congestion.

When serving as executive director of the Chicago (Ill.) Plan Commission in 1941 and 1942, the writer was one of the exponents of an "inner belt" expressway to distribute traffic from all radials. This inner belt utilized the existing Wacker Drive and Outer Drive and provided for completing the circuit. "The Preliminary Comprehensive Master Plan"[20] retains this important feature of the over-all future highway pattern.

Downtown off-street parking facilities for passenger cars are coming to be recognized as intrinsic parts, not only of comprehensive city planning, but of specific actual highway planning, design, and construction. When an express artery skirts close to the business center, parking spaces or garages should be close to the artery. Sometimes, they may be physically connected with it, so that cars can leave the highway and enter a garage without utilizing local surface streets. This arrangement is possible in cases where an elevated expressway (rather than the preferable depressed expressway) is required for compelling reasons.

With the inevitable traffic increases that every city must provide for, over the next 10 to 20 years, it is self-evident that maximum utilization must be made of the existing investment in surface streets, whether or not free-flowing arteries are constructed. Thus, city roadways must be free of parked cars to release curb-side lanes for moving cars; and in the daytime trucks must eventually be loaded and unloaded within building lines.

The writer considers that the great cost of express highways in downtown areas is better justified if these modern arteries are made available for commercial traffic, than if they are reserved exclusively for passenger cars. Commercial traffic includes both trucks and public transportation buses. It is

[20] "The Preliminary Comprehensive Master Plan," Chicago Plan Comm., Chicago, Ill., January, 1946.

argued sometimes that placing passenger cars on the new expressway will eliminate congestion on the old major surface street and make it entirely adequate for trucks. This argument neglects consideration of the delays at traffic lights and the fact that, to the truck operator, time lost is actually money "out-of-pocket"; whereas to the passenger-car driver, it is a case of time saved possibly being money "in-the-pocket."

A cogent reason for permitting buses on urban expressways is that no city (to the writer's knowledge, at least) has enough high-speed public transportation. A new expressway should thus be conceived as an addition to the public transportation network, and not as a competing facility that will encourage car owners to engage in needless driving, in lieu of utilizing the more economical mass transportation system. In appropriate cases the right of way should also include rapid-transit tracks.

If, for local reasons, an interim decision is made prohibiting commercial traffic on a planned express highway, there should nevertheless be no feature of the design that will later preclude a change of policy. Thus grades, curvature, clearances, and structural strength should all be designed on the basis of eventual use by heavy trucks and buses.

Because of the high capital cost of free-flowing express highways, even under favorable conditions of low land price, careful consideration should be given the policy of defraying at least a part of the debt service by tolls. In downtown districts where trips are short and ramps occur frequently, this will not generally be feasible from an operating standpoint. As the average trip lengthens, and with the wider spacing of ramps that is indicated for the outskirts of metropolitan areas, the feasibility of tolls increases. On highways, the driver who objects to a toll will not be required to pay it. He can always penalize himself, if he so desires, by using the slower local thoroughfare.

The author has stressed the great value of economic analysis as justification for proposed projects. Time saved, multiplied by traffic load, multiplied by value of time gives dollars saved. For the majority of passenger-car users, it is believed that the simple saving of time is what will determine their views—or vote—on a particular planned facility. In 1940, the New York City (N. Y.) Planning Commission prepared a city-wide plan of express highways, so located that no important origin or destination center was more than $1\frac{1}{2}$ miles from one of these express routes. At that time Mr. Barnett made many valuable and constructive suggestions. The writer calculated that a trip of 12 miles would mean $1\frac{1}{2}$ miles at each end on surface streets, and 9 miles by express highway. The 3 miles on local streets would take 15 min at an average speed of 12 miles per hr; and the 9 miles would take 15 min at an average speed of 36 miles per hr. Without express highways, one would have to go the entire distance on local streets at about 12 miles per hr—requiring 60 min. With the express highways, the same trip would take only 30 min, thus saving half of the previous running time.

The author has made a noteworthy contribution to the literature of urban highway planning. The translation into construction of the principles he has stated will go far to relieve the critical traffic congestion that faces every major American metropolitan region.

SPENCER A. SNOOK,[21] M. ASCE.[21a]—It is an opportune time to emphasize the need of arterial highways in cities. The detailed requirements of existing data have been stated in papers by Fred Lavis, M. ASCE, and others,[22] and have been in use for a number of years, especially in New Jersey and within the New York metropolitan area. Nevertheless, it seems that arterial highways in most cities have been deliberately avoided, primarily because of cost; and so, by-passes have been built. These served the purpose for the through travel but totally neglected, in most cases, transportation in larger cities of urban commercial and commuter traffic. This problem should be concentrated upon, as usually the cities cannot afford to handle it. Most of the motor taxes come from these areas although the least money has been spent there.

Obviously, congestion of traffic is the lack of free-flowing traffic; and arterial highways, with access at frequent points, will reduce the cost and time for this urban traffic. There are still plenty of useless, time-consuming traffic lights that could be of the actuated type for cross streets.

Elevated highways with proper side roadways do not make a bad appearance, if architecturally treated. They compare favorably in costs with the depressed highway depending upon local conditions. The traffic noise is not as great and the elevation assists the driver in knowing where he is going.

It is always of utmost importance to have the controls of the structural features definitely set, before the detailed plans and designs are started. Frequently plans that have been started are discarded because some controls are changed, and hence the total cost is increased.

The number of public planning agencies, by whatever name, having jurisdiction in one city should be as few as possible as too many cause "jams" in progress because of inability to agree.

LAWRENCE S. WATERBURY,[23] M. ASCE.[23a]—As Mr. Barnett has stated, in the rural areas there is a fairly good system of highways which, despite certain inadequacies, has served enormous traffic movements. However, it has been impossible, in many instances, to maintain these highways properly during the war period, and many of them have been quite heavily overloaded. As a result, it will be necessary to rebuild some of these, to construct additional links in the rural system of highways, and to improve many of them in accordance with modern design standards.

Between most of the large centers of population there are not too many serious delays encountered by traffic traveling over these highways. The cities, however, present quite a different situation. Traffic congestion and suggestions for its relief are in the minds of many people in the urban areas. Everyone is affected by congested traffic conditions—either as pedestrian, driver of a motor vehicle, or as a passenger using the mass-transportation facilities. Traffic congestion is something the average person encounters, in one way or another, causing him delay or inconvenience. He is naturally thinking

[21] Asst. Engr., N.Y.C.R.R., Cleveland, Ohio.

[21a] Received July 22, 1946.

[22] "Highway Economies," by S. Johannesson, McGraw-Hill Book Co., Inc., New York, N. Y., 1931.

[23] Associate Engr., Parsons, Brinckerhoff, Hogan & Macdonald. New York, N. Y.

[23a] Received August 1, 1946.

of improvements and may have suggestions for relief of any intolerable conditions. He brings pressure on various civic groups, with the result that traffic committees in many of these civic organizations do some very constructive and helpful work in preparing plans for the relief of congestion.

As has been stated, congested traffic conditions probably have been a factor in the deterioration of parts of the cities in the United States.

The need for relief in the urban areas has been recognized in the Federal Highway Act of 1944, which earmarked certain funds for use by the state highway departments in the development of modern highway facilities in the urban areas.

A free-flowing artery in the thickly settled city areas costs a considerable sum of money to construct, and the property damage costs are high. Naturally, it is impossible to construct these needed facilities overnight; it is necessary to construct them in stages, spreading the cost over a period of years. The sections that require the greatest relief should have first priority, and in the majority of cases these seem to be the most expensive in cost per mile of highway. In any business in which vast sums of money are to be invested, the investor would first investigate the economic justification before embarking on the venture. Since it is important for highway planners to be certain that they are proceeding in the right direction, they must collect and analyze, carefully, all the data bearing on the problem, and test all the possible solutions.

Mr. Barnett has emphasized the usefulness of a preliminary engineering report for testing these solutions and for the purpose of uniting all interested agencies in the consummation and approval of a plan for an expressway. This certainly should be the first step in express highway planning, since certain general principles of location and design would be covered in such a report.

Prior to any consideration of expressway location, a thorough study should be made of traffic characteristics of the urban areas. This will include not only a determination of the volume of traffic using the present facilities, but also a complete analysis of the origin and destination of this traffic. It will also be necessary to predict what the future pattern of traffic may be; and, when the most suitable location is decided upon, the volume of traffic that may be expected to use the new facility, must be estimated.

The existing land use of the urban area must be studied, and consideration given to future changes that may be expected as a matter of natural development, or desired as a part of the city improvement plans. These changes are certain to affect the future traffic pattern. A thorough analysis of land use in its relation to traffic is required, so that expressway locations will aid in the future favorable development of the urban area and serve the traffic needs adequately. Population studies, present and future, must be made at the same time as land uses are analyzed, as this too will be reflected in the future traffic pattern.

Mr. Barnett has reminded his readers that the factual data regarding the movement and the origin and destination of motor vehicles leave much to be desired and give the highway planner few clues regarding intracity traffic patterns. However, he must utilize whatever data are available, then supplement these with additional traffic surveys and research. The author has men-

tioned one method of securing these data, by home interviews, which is being given some rather severe tests for adequacy and accuracy. There are other methods of making urban traffic surveys which will also yield sufficient data to make forecasts.

It is impossible to suggest a standard method of treatment in the location of expressways which will have a common application in all cities. Each city constitutes a special problem which must be analyzed individually. However, there are certain general characteristics common to most cities. For the most part, cities have grown with no planned development. The street patterns were not laid out or designed for the large volumes of motor vehicle traffic that wish to use them and traffic congestion has been the result.

The nucleus of the city is the central business district. Extending from this area the city has expanded, and people have moved farther and farther out as transportation facilities of various types have been provided. As congestion increased in the central business district and on the various routes leading to it, decentralization of the city began, resulting in smaller business areas being scattered in the outlying regions. This has caused serious reductions in real estate values in the central business district and has resulted in physical deterioration of many properties.

Development of the expressway provides a facility for safe and rapid movement of a larger number of motor vehicles in and out of the business district, and over the routes leading to it. It has the same function as the development of rapid-transit systems in the solution of mass-transportation problems. The location of the expressway should be coordinated with the service provided by the mass-transportation system of the city and in this way will provide rapid transit for those persons using the mass-transportation facilities.

The traffic pattern of the average city generally consists of a flow of vehicles over several routes converging on the central business district. As they approach the center of the city, the traffic volume is increased by the influx of vehicles from the suburban areas along the routes. Congestion is generally at a maximum in the central district but there may be, and usually are, critical points as highways merge, or as cross traffic is encountered.

The expressway should be so located as to serve not only traffic from outside the city but also traffic originating within the city limits. In this way, the maximum benefit may be obtained from the improvement through provision of express service for local residents of the city. An analysis of traffic destination on main arteries approaching most cities shows that a relatively small proportion is through traffic. The expressway, therefore, should be located to provide service for the greatest number of users—namely, those destined for the central districts of the city and originating in near-by surburban and fringe areas. As the expressway passes through the outer regions of the city, it should be located so that the residents of those areas may have convenient access to enable them to use this new facility for express service from points reasonably close to their homes to the central business district.

By so locating the expressway in these outlying residential areas, buses can also use the expressway, and this will result in an improved mass-transportation service. This is an important factor, since mass-transportation facilities oper-

ating in urban and suburban areas provide a sizable share (especially in the larger cities) of the daily travel needs of persons to and from work, schools, stores, and places of worship or recreation.

The ratio of daily transit riders to the urban population shows the extensive use of mass transit by persons in the urban areas in the United States. In the population class of from 100,000 to 250,000, in the years prior to 1940, this ratio was 38%. In 1943 it was 105%, or more than doubled. In the population class of 500,000 to 1,000,000, the ratio was 72% for 1940, and for 1943 it was 145%.

In some cities, the expressway may be parallel to a rapid-transit or streetcar line; it may then be desirable to develop the expressway and improve the mass-transportation facilities as a joint project.

The expressway should penetrate the downtown or the central business district close enough to enable easy distribution to all parts of the area. Naturally, the location through, or adjoining, the central district will require a detailed study of property damage and construction costs. It will also involve an analysis of the effect on the business and property values in the area. Consideration must be given as well to the problem of providing parking space within short distances of the interchange locations on the expressway. Stopping areas will also be required to accommodate the buses using the expressway.

This is a general policy which will apply to most cities. There are certain instances, however, particularly in the case of some of the smaller cities located between two large cities, where the proportion of traffic passing through and beyond the city is far greater than that destined for the center of the city. In such cases, the expressway should skirt the more congested area or possibly by-pass the city completely.

In cities that have rivers flowing through or alongside the central business district, the expressway may be located advantageously along the river. This will usually result in lower construction cost and will simplify the intersection problem. Where such routes can be developed, full advantage should be taken of these natural barriers to cross-traffic movement but, of course, other things being equal, the most direct route is preferable to a circuitous one.

Although Mr. Barnett did not attempt to discuss structural features of the expressway, a few words regarding the location with reference to existing street patterns may be in order. The width of right of way for an express highway will generally require a full city block. In such cases, the present streets on both sides of the city blocks will remain as service roads. The center section will be used for the expressway and any land, not otherwise required, may be used for parking spaces or appropriate landscaping. In the outlying sections, it is preferable to follow the natural ground level as closely as possible. In the event that there are frequent cross streets to be "grade separated," it will often be desirable to introduce a rolling grade, with the expressway successively passing over and under the existing streets to provide grade separation. As the expressway route approaches the more developed sections of the city, it will frequently be necessary to carry the expressway continuously at a separate grade from the existing streets. In any case, elevated structures should be avoided where possible; however, it is a matter for individual treatment in each

city. Although, in industrial sections, particularly where there are a series of railroad tracks at the street surface, it may be preferable to elevate the expressway.

At appropriate points along the expressway, provisions must be made for vehicles to enter or leave. To locate these interchange points properly, a thorough analysis of traffic movement is again essential. In outlying regions, it is not necessary or desirable to provide access at every crossroad. If possible, they should not be closer than from one half to one mile apart, depending on the location of the intersecting main arteries and their relative importance as feeder routes to the expressway.

As the expressway penetrates the central business district, the spacing of interchanges will probably be less frequent than in the outlying regions; they should be provided to give access to the most important streets in the central district.

Interchanges should be designed with adequate deceleration and acceleration lanes so that there is no interference with the main traffic stream by vehicles leaving or entering the expressway. The most satisfactory type of interchange is one in which the vehicles leave or enter the expressway by a ramp running as nearly parallel to the expressway as practicable. These ramps should connect with the service drive rather than make direct contact with a main cross street. In this way, there will be ample opportunity for the speed of the vehicle to be materially reduced from the speed at which it was traveling on the expressway. As a general policy, vehicles would leave the expressway at the interchange nearest their destination; thus, the motorist would drive a minimum distance on city streets to reach his destination after leaving the expressway. If the vehicle is to be parked in or near the central business district, the motorist should be encouraged to select a parking space that is nearest the point at which he leaves the expressway. The parking space preferably should be within reasonable walking distance or about 1,000 ft from the motorist's ultimate destination.

When the expressway location is properly chosen, it will serve bus and truck traffic as well as private motor vehicle traffic. It will provide a route that may be convenient for local city buses and also one which will serve intercity buses. Upon leaving the expressway, the bus should proceed to a terminal located, in so far as is practicable, at a point most convenient for the majority of the patrons. This terminal should be reached by a minimum of travel on the city streets after the bus leaves the expressway.

Consideration should be given to the possibility of developing bus stops along those expressways that are to serve local mass transportation. Under certain conditions, these may be located safely at the interchanges by providing a separate bus-stop lane, but this arrangement must not interfere with the exit or entrance to the expressway or the main traffic stream. By locating these bus stops along the expressway, it would not be necessary for the buses to leave the vicinity of the expressway until they approached the terminal, and much better local service would be provided.

Street and highway systems are public ways, and, as such, should be planned with a view to the accommodation of all types of highway travel or vehicles.

New facilities should be planned to incorporate, to the maximum extent practicable, provision for public transit facilities whenever warranted in the over-all plan based on population and land-use studies. In this way mass transportation will be made more safe, convenient, speedy, and economical for all the public.

The expressway, located so as to serve the freight distribution and industrial areas, should have interchanges at places where trucks may leave the expressway close to the truck terminals, requiring a minimum of travel on the surface streets.

Parking space, including parkway lots and garages for motor vehicles, should be located so as to be most convenient to the final destinations. Consideration should be given to providing parking space that is essentially a part of the expressway, with entrance and exit directly to it, but not so as to interfere with the through stream of traffic. Parking spaces so located would be reached without any travel on the surface streets of the central district.

The location of the expressway is intended primarily to serve the needs of high-speed traffic originating outside the city and passing to and through the congested central business district. In addition, it will serve the traffic originating in the outlying areas within the city limits and provide this traffic with a high-speed facility. Important consideration should be given to circulation within the central district and it is desirable to provide a facility that will assist in good circulation but not at the expense of retarding the expressway movement to and through the district. Careful consideration must be given to the development of the service drives and surface streets of the central business districts which are to serve as the distributors of the traffic after it leaves the expressway.

As stated, it is not possible to prescribe a standard method of treatment for the location of expressways which will provide a solution for all cities. However, the general principles outlined herein should provide sufficient guidance for the proper location and treatment of expressways as the characteristics of each city are analyzed. The facility can then be designed to serve the particular urban area under consideration best.

These principles would be the guide to be used in the preparation of the preliminary engineering report as recommended by Mr. Barnett. As a result of the various investigations to put into practice these general principles, planners would be in a position to make the recommendations for highway facilities which will adequately serve the present and future traffic needs and aid in the favorable development of urban area and region.

BERNARD L. WEINER,[24] M. ASCE.[24a]—A rather complete general description of express highway planning is given in this paper. Considering the purely technical aspects of the problem, there is very little to be added. The paper is timely; one need be only a casual reader of the newspapers to known that the traffic problem has once more reached extremely serious proportions. It can scarcely be claimed, however, that any really courageous effort is being

[24] With Feld and Timoney, Cons. Engrs., New York, N. Y.

[24a] Received August 1, 1946.

made toward its solution. Although the responsibility for this lack of effort lies in many quarters, the highway engineer cannot escape his share of the blame.

The fact that the automobile designer has advanced beyond the highway engineer is not surprising; and, as is usual in such cases, chaos is the result. The automobile industry has indeed made great progress. Changes in fundamental concepts take place exceedingly slowly and it only required thirty years for the manufacturers to realize that they had something which needed a new approach. Real progress in automobile design began when the industry realized that the concept of the "horseless carriage" was obsolete and led only to trouble. At that moment, an entirely new vehicle was born—the present-day automobile—which has changed very little except for "differences in the shape of the tin." Apparently, planners of highways and of other facilities still do not take into account the probability that the motor vehicle is here to stay.

Mr. Barnett writes (under the heading, "Terminal Facilities") that the truck-loading problem will be solved only by a bold approach. It might be added with equal justification that the "bold approach" is needed for all the problems that plague the world. Wherever mankind has made progress, it is a certainty that bold thinking has been done by someone or other; wherever mankind has failed, it is equally certain that bold thinking was lacking. Even worse, it has often been to someone's real or imagined interest to prevent such thinking—history is full of examples.

At that moment, the newspaper editors are "yelling their heads off" that the traffic congestion problem must be solved. They write "Something must be done"; yet, at the same time, they "point with horror" to the rising cost of local and national government and call for a reduction in "normal" expenditures. The problem can be solved, of course, but it will require more than the mere waving of a wand or the passing of a punitive law. It will not be a "costless" solution, but no investment is expensive if it insures adequate dividends. In cities of all sizes, the losses resulting from traffic congestion are so large and so apparent that there is very little doubt that large expenditures are justified to eliminate it.

It would be easy to show that no real attempt has ever been made to solve most problems of any importance that have been insoluble over long periods of time and have reached an acute stage. Characteristically, there has been nothing but tinkering; there has been little effort to reduce the problem to basic fundamentals. In attacking any problem, nothing should be considered so obvious that it is accepted without question.

Admittedly, large costs would be involved if all buildings were required to provide off-street loading areas, but it is also costly to provide elevator and other services. The street floor, however, need not necessarily be used for loading; the lower levels could be utilized. It is not at all beyond the range of possibility to provide elevators large enough to lower and raise entire trucks 10 ft or 20 ft below the street level, at locations where ramps are not possible for lack of space.

There has been altogether too much tendency to think of the express highway as a "study in motion," almost as if it were built for mere aimless motion.

There are three parts to a trip taken by a vehicle: The trip begins with the vehicle at rest at the starting point where it loads; it then proceeds to its destination over local streets and over highways; and it finally comes to rest again at its destination, where it unloads. Unless provision is made for the two end conditions, the express highways may just as well not be built. It is of little value to move a truck or passenger vehicle speedily from one point to another if all the time saved, and more, is lost because it cannot come to a stop quickly.

Traffic control—as distinct from mere law enforcement—is now largely in the hands of the police. It just happened that way. This control is so complete that the various police forces of the United States virtually have legislative powers. The police's point of view is punitive and only in rare instances is it constructive. "Hounding" the motorist and fining him, as much as fifteen dollars for parking in New York, N. Y., may achieve the result of keeping cars out of the congested areas; but it is no solution (see heading, "Radial Highway Arteries Are the Greatest Need: (3) Relief of Congestion"). The people do not receive the benefit of the use of their cars and, in a city like New York, the curse of centralization has made the public transportation system unfit for human occupation.

Better results could possibly be obtained if the police were restricted entirely to the enforcement of the law as interpreted and amplified (by the necessary administrative regulations) by an entirely independent body. Such a body should be made up of representatives of the various interests involved and should be under the general supervision of competent traffic and highway design engineers. It should be well financed, and the personnel should give its full time to the job of traffic control.

The purpose of this body would be to study ways and means of facilitating the movement of traffic in all its phases. Such studies would include major improvements, but comparatively minor changes would not be neglected either. Most important of all, since such a body would be made up of civilians, it would be more likely to keep in mind that the purpose of all regulations is not the punishment of violators but keeping traffic moving smoothly—and to permit it to stop where it legitimately should stop.

As stated, the police's point of view is rarely constructive. When the costly approach to a major bridge in New York City was opened to traffic, it was found that a combination exit to, and entrance from, a service road created a traffic hazard. As it happened, the exit was important, for it permitted traffic from a main thoroughfare to reach an existing bridge across the Harlem River, directly and quickly. The police "solved" the problem by closing the exit, thus vitiating an important part of a costly improvement. As it happens, a rather inexpensive piece of reconstruction would have eliminated the hazard and also saved the exit. Although it is quite a few years since the exit was closed, nothing has been done about it. This example is typical, however, of the police methods of controlling traffic.

Many traffic lights could well be eliminated; others can and should be synchronized. Also, more traffic controlled lights at minor intersections could be used to good advantage. The motorist is also too familiar with police signs reading, "No Parking Between Signs." It is a common experience to find such

signs around a bank or main post office in a large city. Common sense would indicate that a reasonable parking period—from ten to fifteen minutes—be permitted to allow for the average business transactions at such places. Although this is done in small towns, the contrary is true in large cities—even in residential districts. The motorist would be more inclined to cooperate if it were evident to him that an attempt was being made to give him every possible "break." As it is now, rules and regulations are ignored and those who have legitimate and necessary business in restricted areas charge parking tickets to operating expense—when they are caught.

In his discussion of the deteriorated areas along the approaches to cities, Mr. Barnett decries the fact that obsolete laws and pressure groups prevent the protection of traffic facilities—a condition which, he states, is not the fault of the highway engineer. Issue might be taken with the last part of this statement. Not only in this particular case but also in the more general social-political-economic problems of the nation, the engineer is altogether too prone to consider his professional responsibilities from too narrow a point of view—that of a mere technician. It seems to the writer that it is as much the job of the engineer to educate the public, study the financial aspects of public improvements, etc., as it is his job to perform the actual technical duties.

The civil engineer has too long considered himself, and has also allowed others to think of him, as a spender rather than as a producer. It would be a good idea, in fact, to adopt or invent a new word for "taxes" as applied to public works—for the word "tax" originated in the days when people paid tribute for mere physical protection. In the matter of highways, the taxpayer should be made to realize that, just as he pays for his car and its maintenance, he must also pay for the highways, which are essential parts of his motor vehicle transportation system. Industry is becoming more and more dependent on public relations; why should not the professions likewise become dependent?

In passing, it should also be stated (see heading, "Radial Highway Arteries Are the Greatest Need: (2) Shelf of Plans") that the sooner the idea of using public works "to provide jobs" is abandoned, the better for the profession and for the nation. Public works should be built only because there is a need for them in their own right. Public works cannot cure unemployment, and any attempt to make them do so leads, sooner or later, to "make work" projects at starvation wages—which does no one any good.

Like Mr. Barnett, the writer does not wish to raise the subject of financing; but one observation may be made. Although the author expresses the "orthodox" and accepted "financial habits" in use today, the present method of financing by borrowing is an anachronism. The bold approach might well be applied to the financing phase of all projects—public works and other construction. It is well known that public works financed by borrowing cost eventually two and one-half dollars for every dollar originally borrowed. With a bold approach to this problem, this excessive cost could be largely eliminated.

It is an accepted truism that a bridge destroys the value of property whereas a tunnel improves it. This maxim could well be applied to the express highway. The viaduct should never be used except as a last resort. Furthermore, sound engineering takes all factors into account—including human nature. Man—in

spite of his vaunted civilization—still likes the good earth, and being forced to ride above it is not pleasant. Also, from a more practical point of view, viaducts are never wide enough to provide for the inevitable disabled car. A single accident can, and does, block traffic for a mile and more back. From these and other points of view, the viaduct is undesirable.

Finally, in relation to expressways and parkways in general, there is another item to be mentioned. The landscaped and separated expressway tends to approach perfection—and, oddly enough, for this very reason tends to become monotonous even for comparatively short distances. For long distances—50 miles or 100 miles or more—the "perfect" highway does become deadly monotonous. The answer to this problem is difficult to define; but, nevertheless, highway engineers should give it their attention. The customer, after all, must be pleased.

George H. Herrold,[25] M. ASCE.[25a]—The difference between rural and urban routes and their capacity to serve the traffic using them is caused by the difference in population density in rural and urban areas, and by the fact that the states have large sums of money to expend freely in rural areas whereas such funds have not been available to the cities. As an example, the ninth Federal Reserve District (comprising Minnesota, North Dakota, South Dakota, and part of Wisconsin and Montana) has an average density of population of 13.5 persons per sq mile, compared with the cities of St. Paul and Minneapolis, Minn., which have an average density of 8,000 people per sq mile.

Before World War II, the United States was greatly exercised over traffic congestion and the decentralization of business districts. This decentralization or disintegration was caused by hazards created in the central business district by motor vehicles, by high speeds, and by poor law enforcement. Careful and timid people gradually stopped coming to the central business district unless they could not avoid it. Instead of being an attractive place to visit, the central business district became an undesirable place, with a corresponding reduction of business income and a lowering of taxable values.

During the war period people discovered that the transit industry was geared to handle the people who had to move from one place to another. Motor vehicle transportation decreased because of cars that broke down and because of restrictions in their use; and mass transportation increased by leaps and bounds. However, to a large extent, the congestion problem—the congestion that occurs at regular times and definite places—was solved. In St. Paul the number of people that used automobiles to take them to work decreased 47% whereas the number of people traveling on street cars and buses, with the aid of staggered working hours, increased accordingly.

When state trunk highways were first designated, the routes selected were existing highways leading to the city and its business district. These routes were chosen because they best represented the travel habits of farmers going to market or people from small towns going to the city. As a rule, the highways were improved only to the city limits, leaving to the city the problem of

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[25a] Received July 15, 1946.

handling this traffic, which was augmented by the traffic generated in the city. Traffic from these trunk highways used the city street system to reach destinations—various "points" in many areas including the central business district.

A city is laid out in blocks bounded by streets designed to make the blocks accessible and usable. On these blocks are buildings and people, and each block has its quota of motor vehicles. Traffic finds its way on cross streets to a trunk or arterial highway, and there it merges with the traffic from outside the city and proceeds toward destination areas and the central business district. As the traffic from outside the city moves toward the city's center, it merges with the traffic from the city's cross streets which are predominately for serving property owners.

Under the heading, "Radial Highway Arteries Are the Greatest Need," the author states:

> "It has become clear, also, that traffic congestion in cities will be solved to the greatest extent feasible by the construction of facilities that permit the uninterrupted flow of traffic into and through the cities."

Mr. Barnett cites three types of expressways: The expressway at grade—that is, built on the surface of the ground; a depressed expressway, and an elevated expressway. No matter which type is adopted, the expressway will be used by the same or increased volume of traffic, and the traffic will reach the central business district at the same time that it now arrives there. Instead of the city-generated traffic entering the expressway at every street intersection, a number of streets would be closed off and the traffic would enter the expressway every mile or so. Instead of one vehicle per minute entering the expressway at each street intersection there would be, say, sixteen vehicles per minute entering at designated access points. On an expressway these vehicles could travel at greater speed than on other routes. The driver could leave home 5 min or 7 min later, but would arrive at the central business district at the same time he arrives there now. Although an expressway does not solve any congestion problem, it increases speed and removes some frictions; and it moves people faster to the areas of congestion.

Fig. 12 shows the origin and destination study made by the Minnesota Highway Planning Survey in the St. Paul-Minneapolis metropolitan area. It shows the traffic flow in cities of St. Paul. (hatched) and Minneapolis, (stippled) that originates in the satellite areas of the two cities. Each city has its distinct following and the flow from one to the other is negligible. The cordon where the interviews with motorists took place is about 1.5 miles outside the twin-city limits. In Fig. 12, the numbers within the hatched and stippled areas represent the average daily traffic observed between 6:00 a.m. and 10:00 p.m. in July and August, 1941. A traffic flow of less than 400 vehicles per day is shown by a single line and a number.

It is extremely important that "trading behavior"—the social habits and the economic reasons behind them—be studied in determining the location of an expressway and its access connections and exchanges. The intracity traffic flow presents a different picture. In St. Paul the wide traffic flow bands run

FIG. 12.—FLOW PATHS OF TRAFFIC THAT ORIGINATES IN THE SATELLITE AREAS OF ST. PAUL AND MINNEAPOLIS, MINN.

east and west. In Minneapolis they run north, south, and southwest, illustrating that the intracity motor traffic is predominantly for going from home to work (or vice versa), distributing commodities, shopping, and visiting—either for business or for pleasure. The wider bands on certain streets indicate a traffic excitor—people moving two, three, or ten blocks to contact one another. There is only an occasional through movement. The buildings along the streets and the people in them cause these motor vehicle movements.

The writer is apprehensive as to the effect of arguments or inferences that expressways solve congestion problems. The true problem that confronts all cities is one of mass transportation—to devise a well-balanced movement of people and goods.

It makes no difference (except as to the cost) whether the traffic is taken to congested areas on a limited-access surface street, a depressed limited-access highway, or an elevated expressway. Where a block of land plus two streets is to be acquired for construction, there are great advantages in the elevated highway—economies as to construction, economies as to the use of the land, economies in utilities, and economies in maintenance.

Mr. Barnett is correct in stressing the advantages of a close-in circumferential route and also (see "Use of Existing Streets") in stating that a complete plan is much more than "bulling" through a free-flowing type of highway. Fig. 3, showing the moving of an arterial road from the central business district to the riverbank, illustrates good location. In a number of places the author makes use of expressions such as "a flexible system of expressways," "flexible routes for the driver to choose from," etc. In a city things are fixed—buildings are erected, utilities constructed, and street systems are fixed in relation to permanent transit systems. The wealth of a city has been built up around these fixed things. The word "flexible" is extremely misleading.

Mr. Barnett writes (under the heading, "Planning Agencies") of "* * * a spectacle of so much planning that what should have been a healthy rivalry resulted in a jam which prevented progress." Possibly some one was stubborn or had a limited technique. However, cities are full of things called "progress" that were put there before all the facts were known. There are many activities in a city other than moving around in an automobile.

In the "Introduction," the author states that "As yet there have been few instances in which the desirable separate provision for arterial traffic has been attempted." The business transacted in a çity, the visiting, and the contacts made are not from arterial traffic comparatively speaking. The converging people from 5,000 blocks of 50 sq miles of city make business; and, if these people come by automobile, congestion results.

It seems confusing to state (see "Use of Existing Streets") that:

> "One fallacy in such reasoning lies in the fact, just mentioned, that an individual vehicle is destined for a point in a district and use of surface streets should be reduced to a minimum."

Each vehicle is destined to a point and every point is a different location that can be reached only by surface streets. The existing major streets must accommodate the bulk of the movement of people and goods.

An express highway through a city requires land, approximately 400 ft wide, with ramps, service roads, roadways, and accelerating and decelerating lanes. Except where stragetically located, such a structure introduces a disrupting force to all the factors of good living. If a school building is on one side of this expressway, the children who live on the other side must go to one of the arterial crossings and backtrack to their school building instead of going the usual route directly from home to school; or there must be a new arrangement of school districts and new school buildings. Because they must carry all transportation of the closed streets in between, these arterial crossings become greater hazard factors than before. People going to church are subjected to the same inconvenience, and neighbors cannot visit neighbors as before. They cannot shop at their favorite neighborhood store or use their favorite neighborhood service station. For the city as a whole—school districts, church districts, social districts, and neighborhoods that may be cut in two by an express highway—the effect on the city tax structure may be an injury that cannot be compensated.

Mr. Barnett's paper concerns the Interregional or Interstate Highway System as extolled in *House Document No. 379* (2d session, 78th Congress). This system is a network of highways connecting the main cities of the nation, apparently designed for military routes. However, it is to be used in peacetime for all-purpose traffic, trucks, trailers, semitrailers, buses, and passenger automobiles. The standards of construction are to be the highest standards used in highway construction. The highways are to be designed for a speed as great as 70 miles per hr and will be expected to care for all motor traffic that is likely to be developed in the succeeding 20 years. The system will enable buses and trucks to compete effectively with railroads. It will bring these buses and trucks directly to the heart of the city, through the city, and beyond the city. As compensation to a city, it is stated that these express routes running through the business center, and through the city, will serve the population of the city between home and business. This conclusion is extremely doubtful.

The writers of *House Document No. 379* maintain that the shortest route may not always be desirable and that the most direct route should give way to locations which would not be disrupting—riverbanks, waterways, and railroad rights of way (where the street system has already been broken and where all development of the city in past years has been based on this situation). An express highway, if one is needed through a city, should be located where it will disrupt the cross street system the least. Surface streets are a necessary part of any comprehensive system of transportation. The traffic picture in a city is changed continually by the location of new buildings, new factories, new manufacturing districts, and new residential districts. Therefore, a plan must be adopted in which the express highway does not cut across streets that have been developed, to the fullest extent through the years, to serve the abutting property. In this case, location is far more important than engineering design, and it must not be thought that, if the location destroyed a few old buildings, it is necessarily a good location.

The problem confronting all cities is one of transporting people. An express highway is built for the use of passenger automobiles and all other motor vehicles—motor buses, motor trucks, and tractor-trailer and semitrailer com-

binations. It is not for the much needed segregation of passenger automobiles and the rapidly growing truck traffic.

The great spread of destinations of passengers from all parts of the city through the entire central commercial area, the manufacturing area, the wholesale area, the railroad terminal area, and the waterfront area requires further exploration. A check on where people alight from street cars and buses furnishes a good sample of this phenomenon. The people's habits and behavior, as they move from place to place, involve a surprising number of tangible and intangible factors, all of a variable but interrelated nature.

To expend from three to five million dollars per mile for an express highway into and through a city is a major operation, and location is all important. Large cities are prohibiting parking of cars in the central business district, establishing one-way streets, and exploring methods of "steady flow traffic" to make greater use of their existing street system. All these studies and experiments are valuable. Possibly the "close-in circumferential route" should be located and built first. The traffic flow into it would help locate radials and interchanges.

No passenger cars should be allowed inside the circumferential route. People would transfer from automobile to shuttle buses and taxicabs, or they would walk to their destination. From an economic standpoint it would be far better to improve mass transportation to the highest standards possible, and then later to follow up with motor vehicle express routes. Building express routes first will increase congestion to a self-limiting status. Building up mass transportation will move the people where they need to go and may stop disintegration of central districts.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

EFFECT OF STRESS DISTRIBUTION ON YIELD POINTS

Discussion

BY GEORGE WINTER, AND O. SIDEBOTTOM AND T. J. DOLAN

GEORGE WINTER,[6] M. ASCE.[6a]—The question raised in this paper is of considerable interest in connection with modern tendencies toward what is variously called "limit design," "ultimate design," or "plastic design" (32).[6b] The application of this method to statically determinate and indeterminate problems of flexure is essentially predicated on the "old theory" of plastic stress distribution as given in Fig. 1. Problems of column buckling beyond the elastic range are also analyzed fundamentally by the same concept with good experimental confirmation (33). If the author's opinion, that this type of stress distribution does not actually occur, were confirmed by sufficient evidence, substantial revision of present ideas on ultimate design of steel structures would become necessary.

The problem is difficult to approach experimentally, chiefly because, beyond the yield point, the ordinary kind of strain measurements can no longer be translated into stress. Even if the strain distribution were investigated carefully, therefore, no conclusion could be reached on the absolute magnitude of the stresses in the "overstrained" parts of the beam.

The difficulty can be overcome to some extent by X-ray stress measurements. This method measures changes in distances of lattice planes in the crystals, but is not affected by relative shifts of various parts of the crystals due to "sliding-yielding." Therefore, it measures that part of the strain which is proportional to stress. The accuracy of this method, however, is generally of the order of from 5% to 10% of the yield point of mild steels. It has not attained the same degree of dependability as strain measurements, therefore, although effects of the order of 40% of the yield point, as indicated in the paper, are easily detected by this method.

NOTE.—This paper by F. G. Eric Peterson was published in April, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1946, by F. P. Shearwood, Edwin H. Gaylord, and L. J. Mensch.

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[6a] Received July 8, 1946.

[6b] Numerals in parentheses, thus: (32), refer to corresponding items in the Bibliography (see Appendix of the paper), and at the end of discussion in this issue.

In addition to such X-ray measurements the actual stress distribution can be inferred by indirect methods, such as the occurrence of Lueders lines, permanent distortion, magnitude of curvature and deflection, and similar secondary effects.

That the situation, fundamentally, is not so simple as usually assumed can be recognized from the following consideration: For steels in the elastic range Poisson's ratio is from about 0.25 to 0.30. On the other hand, in the plastic range Poisson's ratio should be that of a liquid—that is, 0.5. Consequently, at the transition from the elastic to the plastic range, the lateral deformation would have to increase suddenly and discontinuously by from 60% to 100% of its maximum elastic value. In other words, the cross section of a rectangular beam stressed partly into the plastic range should be of the shape shown (exaggerated) in Fig. 12(*b*). Such a shape can develop only if the shear resistance in the outer parts of a-a is completely overcome. Actually, a shape more like that of Fig. 12(*c*) must be expected. It is clear that this shape causes shear

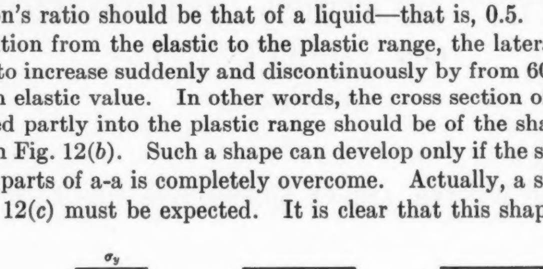

FIG. 12.—DISTRIBUTION OF LONGITUDINAL STRESS AND OF LATERAL STRAIN IN BEAMS STRESSED BEYOND THE YIELD POINT

stresses at and around section a-a such that at least part of the beam above a-a is subject to lateral compression, and part of the beam below section a-a is subject to lateral tension, in addition to the primary, longitudinal compression stress. At and near the supposed boundary of the elastic zone a two-dimensional state of stress, instead of the assumed uniaxial condition, is obtained. Assuming the yield condition to be defined by the distortion energy theory, these lateral stresses cannot fail to affect the yield point of the material in that zone.

From this consideration alone it is evident that the "old theory" can be expected to represent only a reasonable approximation to the actual condition. In addition, Fig. 12 suggests another possible experimental approach for investigation of this problem, namely—the determination of Poisson's ratio by test. It is relatively simple to make simultaneous longitudinal and transverse strain measurements on the top surface of beams of any cross section, for example, by means of electric-resistance gage rosettes. If properly evaluated, the stage at which Poisson's ratio increases from its elastic value and, finally, reaches 0.5 (most likely) would give at least some indication of the actual condition of the metal—plastic or elastic.

A review of existing evidence, alongside the author's interesting tests, is rather perplexing; but it shows, at least, that the situation is not so simple as indicated in the paper. In the following survey of available information, the

writer purposely omitted investigations based on the appearance of Lueders lines as evidence of yielding. These lines seem to be a satisfactory indication of the occurrence of strains in the specimen (beam) of magnitude equal to that which, at the yield point, occurs in a tension specimen. However, in the case of nonuniform stress distribution, the strains in "overstressed" zones adjacent to parts that are stressed in the elastic range are necessarily of the order of magnitude of elastic strains. It is doubtful whether macroscopic Lueders lines can be detected when yield strains are of such small amount, although it is likely that slip bands in individual crystals could be observed in such zones by microscopic inspection.

In an extensive investigation (34) E. Volterra obtained interesting experimental evidence on the behavior of a great number of "overstressed" steel beams. Although the report of this work is somewhat sketchy and contains minor errors (34*a*) it presents broad evidence that W. Kuntze's and W. Prager's (35) ideas are not confirmed by test. (Messrs. Kuntze and Prager have held that there is a sharply increased yield point in the "overstressed" parts of a beam; that there is purely elastic behavior far beyond the tensile yield-point stress; and that, subsequently, there is a sudden breakdown of elasticity in the entire cross section.) On the other hand, Mr. Volterra's tests also present evidence that, in the "elasto-plastic range" (that is, at loads larger than those causing tensile yield-point stress in the outer fiber, but below the ultimate), the curvatures and deflections are smaller than those predicted by the "old theory."

Both these facts are well illustrated by two curvature-bending-moment diagrams (Fig. 13) taken from the Volterra paper. Mr. Volterra attributes the difference between measured and computed curvature to the effect of strain-hardening, whereas, in a discussion of that paper J. F. Baker, Assoc. M. ASCE, (34*a*), holds the upper yield point to be responsible for that difference, a factor which is generally neglected in the "old theory." In the writer's opinion the phenomenon depicted in Fig. 12 is likely to represent at least a contributing factor to this limitation of deformations to values below those expected from the "old theory."

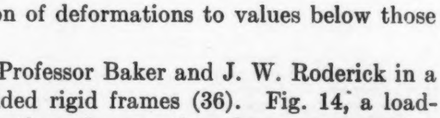

FIG. 13.—LOAD-CURVATURE DIAGRAMS FOR TWO STEEL BEAMS ACCORDING TO E. VOLTERRA

Similar results were obtained by Professor Baker and J. W. Roderick in a very important investigation on welded rigid frames (36). Fig. 14, a load-deflection diagram from this investigation, shows rather close agreement between experimental values and those computed by the "old theory," again exhibiting a somewhat sharper curvature of the load-deflection curve than predicted theoretically.

Whereas measurements of deflections, curvatures, and observations of Lueders lines give only indirect evidence of the actual stress distribution in a beam—X-ray measurements, within their limits of accuracy, enable the investigator to determine the magnitude of stress directly, in the elastic and in the plastic range. In this connection F. Bollenrath and F. Schiedt (37) observed

(see Table 6) that in the period from 1932 to 1938 successive investigators found less and less difference between yield points in bending and in simple tension.

These tests relate to rectangular sections, although the last of these investigations was extended to triangular and other shapes. Messrs. Bollenrath

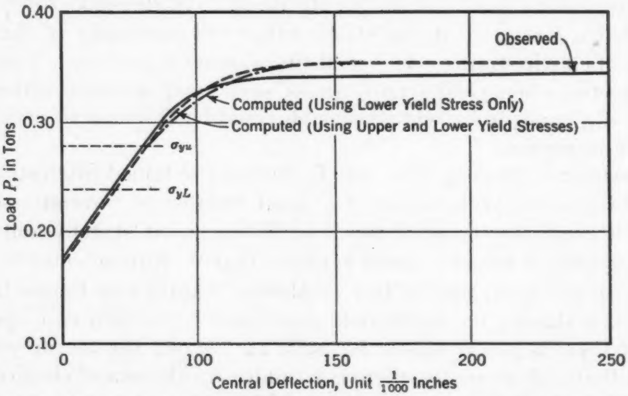

FIG. 14.—LOAD-DEFLECTION DIAGRAM FOR A STEEL BEAM ACCORDING TO J. F. BAKER AND J. W. RODERICK

and Schiedt stated their conclusion as follows (37*a*): "Independent of the shape of the cross-section no increase of the yield point does occur in bending as compared with that determined from a tension test."

Similar conclusions are reported in papers recently published in the United States by J. T. Norton, D. Rosenthal, and S. B. Maloof (38)(39). In the first of these investigations (38) X-ray stress measurements were made in notch-bend tests—that is, on specimens that have steeper stress gradients than ordinary, prismatic beams and, consequently, in which the supposed increase in yield point should be larger than in such beams. However, in the words of Messrs. Norton, Rosenthal, and Maloof (38*a*) "* * * the X-rays reveal no increase of yield point as a result of notching or steep stress gradient."

TABLE 6.—COMPARISON OF PERCENTAGE INCREASES IN YIELD-POINT STRESS

Investigators	Year	Increase	Criterion
A. Thum and F. Wunderlich (2)	1932	35 to 40	Lueders lines
H. Moeller and J. Barbers (40)	1934	40	X-ray
E. Siebel and F. H. Vieregge (16)	1934	28	Beam deflection
H. Moeller and J. Barbers (41)	1935	13	X-ray
F. Rinagl (42)(43)	1936	0	Residual strains
F. Bollenrath and E. Schiedt (37)	1938	0	X-ray

It cannot be stated that these few quotations shed any definite or final light on the actual process of yielding in nonuniformly stressed members. A much more extensive review of available evidence than can be discussed in this brief contribution convinced the writer that the actual details of the process depend, to some extent, on type, treatment, and structure of the steel, and other influences. That yielding is not a process of continuous flow, but that it occurs in discontinuous, although minute, layers also appears (44) well estab-

lished. The dimensions of these layers, as compared with those of the member or those of the part of the member, subject to nonuniform stress, also may influence the over-all behavior beyond the elastic range.

However, the structural engineer is little concerned with these details. In deciding the rather vital question of practical reliability of the newly proposed methods of limit design, plastic design, or ultimate design, he is chiefly interested in final results—that is, whether or not carrying capacities and deformations determined by these methods agree with test results. In this respect it is noteworthy that the "old theory," as well as the theories of Messrs. Kuntze, Prager, and others, agrees closely in reflecting the influence of cross-sectional shape on ultimate moment, and that these computed ultimate moments are extensively verified by test. Also noteworthy is the fact that deflections and curvatures measured by different investigators were always found to be either equal to, or somewhat smaller than predicted by, the "old theory." Hence, this very simple and practical theory seems to be well confirmed with regard to ultimate strength, but errs slightly on the conservative side with respect to deformations beyond the elastic limit. In the field of statically indeterminate structures, where this design method is particularly appropriate, the degree of agreement of test results with values computed by the "old theory," for seven-portal model steel frames (45), is evident from the following:

Portal No.	Failure Load, in Tons: Computed	Observed
F.3	1.54	1.60
F.4	1.45	1.64
F.5	1.54	1.62
F.6	3.08	2.98
F.8	2.49	2.79
F.9	1.58	1.64
F.10	1.65	1.75

With one exception, discrepancies do not exceed 6%; in the one case of lesser agreement (F.8), the "old theory" errs on the conservative side by about 11%.

Despite the fact, therefore, that the details of the yielding process are probably not so simple as frequently assumed, it is this writer's opinion that the "old theory," by present evidence, represents a simple and reliable tool for structural analysis.

Acknowledgment.—Some of the quotations cited in this discussion were brought to the writer's attention by N. M. Newmark, and Bruce G. Johnston, Members, ASCE.

O. SIDEBOTTOM,[7] ESQ., AND T. J. DOLAN,[8] ESQ.[8a]—Structural designers usually have assumed that the material in a ductile metal member subjected to uniaxial stress starts to yield (or attains a small measurable plastic deformation) at a given stress, regardless of the existence of a stress gradient on the cross section of the member. In recent years, however, the results of several in-

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vestigators (see for instance, J. L. M. Morrison (46), G. Cook (47), Fujio Nakanishi (48), and G. Brewer (49)) and the tests reported by the author have been interpreted as showing that yielding is suppressed in the presence of a stress gradient and hence that an "elastic" peak stress can be built up which is greater than the yield point developed by the material under a uniform stress.

The finding of the author (see "Conclusions") that "The stress at which yield occurs is higher under nonuniform stress * * *" than that found from axial tensile tests, in which the stress is uniform, does not agree with the conclusions reached in studies in 1946 by D. Morkovin and O. Sidebottom (50), and by J. T. Norton, D. Rosenthal, and S. B. Maloof (38). It seems desirable, therefore, that a review be made of the fundamental structural actions involved in a beam to clarify some of the divergent conclusions of various investigators. The writers feel that the conclusions of the author create false impressions which, if not reinterpreted, may lead to unsafe design practices. The material presented in this discussion is based on the work of Messrs. Morkovin and Sidebottom whose study included a re-examination of the data of several previous investigators.

The reported changes in yield point for a given metal under different test conditions, involving stress gradients in uniaxial stress, may be attributed to two factors: (*a*) Difficulties in detecting the load at which yielding started in the most stressed fibers, resulting in erroneous interpretation of the test data; and (*b*) the varying stress levels at which the upper yield-point phenomenon may be exhibited by certain mild steels under different test conditions. The following discussion will deal mainly with a study of item (*b*) since the writers believe the material in the members tested by Mr. Peterson exhibited an upper yield point. However, item (*a*) will be considered first in order to facilitate the explanation of item (*b*).

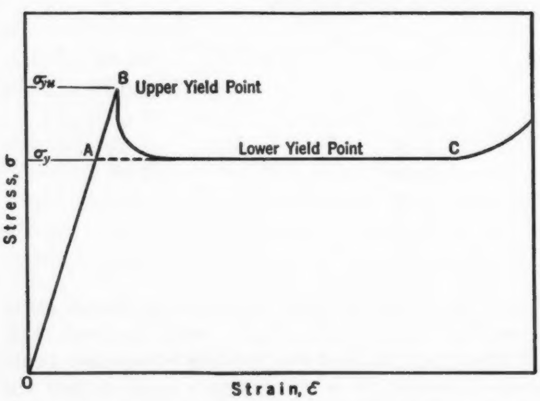

FIG. 15.—STRESS-STRAIN DIAGRAMS FOR TENSION SPECIMENS

First, examine the resisting moments that would be developed by a beam if each fiber exhibited exactly the same stress-strain relationship as that determined for a tension (or compression) specimen. By assuming that, in a beam, plane sections remain plane, and the tensile and compressive stress-strain diagrams of a material are identical and similar to curve OAC in Fig. 15, it is possible to develop mathemathical equations based on equilibrium conditions relating the bending moment in the beam to the strains, ϵ, developed in the extreme fibers for various cross-sectional shapes. The curves shown in Fig. 16

have been so derived (50), and have been plotted in terms of the dimensionless ratios M/M_y and ϵ/ϵ_y to facilitate comparison for the five cross sections illustrated. In this discussion the symbols M_y, σ_y, and ϵ_y will designate the bending moment, stress, and strain, respectively, corresponding to the lower yield point of the material; M and ϵ represent the bending moment in the test part and the strain in the extreme fiber, respectively, developing at any given stage of the test.

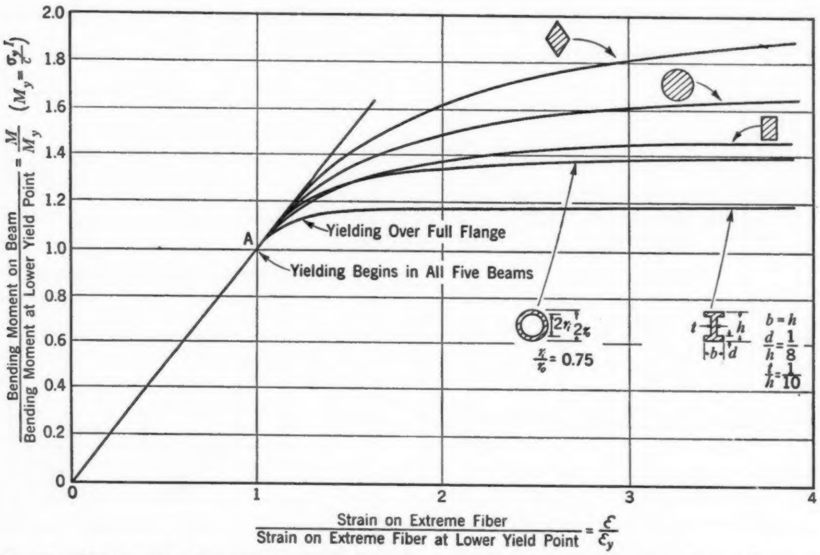

FIG. 16.—THEORETICAL MOMENT-STRAIN DIAGRAMS FOR METAL WITH A DEFINITE YIELD POINT, σ_y (STRESS-STRAIN DIAGRAM SIMILAR TO CURVE OAC IN FIG. 15)

As the relative volume of material in the extreme fibers is decreased by modifying the shape of the beam (see Fig. 16), there is a tendency for the departure from the initial straight line to become less abrupt. This leads to an illusion that the proportional limit has been raised. Actually, however, all curves begin to depart from the straight line when the ratio $\frac{M}{M_y} = 1$ is exceeded. Although all curves may represent identical material, the bending moment at which the deviation from linearity becomes a definite measurable amount (as evidenced also by distortion of the member as a whole) is a function of the shape of the beam. Fundamentally, the resisting moment developed by a beam and plotted in a moment-strain curve represents an integrated sum of the resisting moments developed by all fibers of the member. Thus, although yielding in a beam starts in the outer fibers at the same stress, the potential load-carrying capacity of fibers beneath the surface makes it much more difficult to detect initial yielding experimentally than is the case in a simple tension member.

Comparisons of experimental moment-strain data for rectangular beams obtained by Messrs. Morkovin and Sidebottom (50) with theoretical curves of

the type shown in Fig. 16, are presented in Fig. 17. In Fig. 17(*b*) good agreement is seen to exist between the moment-strain diagram predicted by the "old theory" and the actual test data for two beams made of an annealed high-carbon steel. In analyzing the data shown in Fig. 17(*a*), it was found that for two of the mild-steel beams initial yielding began at the lower yield point as determined from tensile tests; whereas, the third mild-steel beam (shown by solid circles) exhibited an upper yield point approximately 10% greater than the tensile (lower) yield point. As plastic action progressed, subsequent

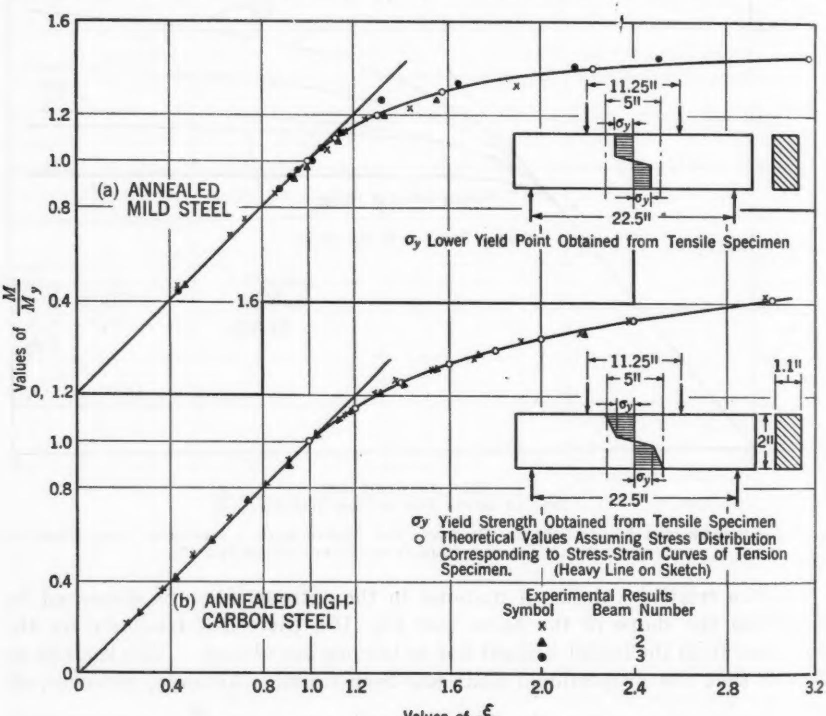

Fig. 17.—Comparison of Theoretical and Experimental Moment-Strain Diagrams for Rectangular Beams

yielding of the mild-steel beams occurred at a stress equal to the lower yield point. (Analyses of these data in terms of stress were facilitated by use of equations developed by H. Herbert (51).) Thus, for cases in which an upper yield point was not exhibited, the stress gradient occurring in a beam did not raise the yield point of the material. For materials with a high upper yield point the theoretical curves of Fig. 16 must be modified to provide for this phenomenon.

The occurrence of a high upper yield point in carefully conducted tensile tests is a phenomenon long associated with mild steel and is a factor which, if present, will modify the behavior of a beam in the early stages of yielding. If special precautions are taken in tensile tests, the stress-strain diagram of the

material may be of the shape OBC as shown in Fig. 15, in which an upper yield point, σ_{yu}, is exhibited. The magnitude of the upper yield point is extremely variable and unpredictable depending upon such factors as (*a*) the rigidity of the testing machines; (*b*) the alinement of the specimen in the grips; (*c*) the radius of fillets on the specimen; and (*d*) inherent stress raisers such as surface roughness, residual stresses, or discontinuities in the material. If the effects of these variables are minimized successfully, the magnitude of the upper yield point may be considerably greater than that of the lower yield point, σ_y. For instance, G. Cook (47) has obtained values for upper yield point as large as 1.56 σ_y.

If a mild-steel tension specimen can be tested under such favorable conditions that an upper yield point will be exhibited, it is reasonable to assume that the material in the most stressed fibers of a mild-steel beam, tested under similar conditions, will exhibit an upper yield point. Furthermore, the magnitude of the upper yield point exhibited by the most stressed fibers of the beam will be larger, in most cases, than the value obtained from the average stress in a tensile test. In the foregoing paragraph four factors were listed which tend to lower the upper yield point in a tension specimen. None of these factors (except perhaps inherent stress raisers) has as pronounced an effect in lowering the upper yield point in a beam as it has in a standard tension specimen.

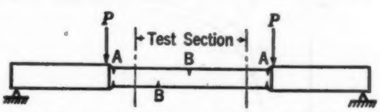

FIG. 18.—YIELDING IN TEST SPECIMEN

If the bar shown in Fig. 18 is subjected to an axial tensile load, there are no available understressed fibers to carry the load by redistribution of the stress once yielding is initiated. Thus, the stress concentrations existing at the fillets cause the average stress at which yielding starts in the tension member to be lower than the maximum upper yield point actually developed at the fillets. Subsequent yielding in the tension specimen will progress by branching of the plastic zones throughout the body of the test section at a stress equal to that of the lower yield point on the material. Hence, tensile test data as generally obtained do not furnish an accurate measure of the maximum upper yield point developed in the material, and the results are more likely to indicate a value close to the lower yield point. Conversely, in a beam, the redistribution of stress caused by initial yielding in the regions of stress concentration (at point A, Fig. 18) enables the beam to carry greater loads before the plastic zones progress by branching out, or before they start to develop in the test section or part. Several investigators have interpreted this to mean that higher stresses are developed in the beam than in a tensile specimen before yielding is initiated, whereas the increase is mainly due to inherent differences in test condition and lack of knowledge of the exact stresses existing at fillets, load points, or other stress raisers.

The effect of an upper yield point on the behavior of a beam can best be shown by considering the moment-strain diagrams for a rectangular beam shown in Fig. 19. If the inherent stress concentrations are of sufficient magnitude so that none of the material in the most stressed fibers of the beam finds it possible to exhibit an upper yield point, initial yielding will begin at the

bending moment M_y corresponding to the lower yield point $\sigma_y \left(\frac{M}{M_y} = 1\right.$ as shown at point A in Fig. 19) and the moment-strain diagram will follow the theoretical curve OADB which is the same as that shown in Fig. 16. On the other hand, if the inherent stress concentrations are sufficiently small the moment-strain diagram will remain linear until the stress in the most stressed fibers reaches the upper yield point of the material. For instance, if the upper yield point is equal to 1.3 σ_y, initial yielding will start at a bending moment of $\frac{M}{M_y} = 1.3$ as shown at point C in Fig. 19. Unlike the tensile stress-strain diagram in which there is a sudden drop at the beginning of yielding from an upper yield point to the lower yield point, the moment-strain diagram shows no drop when initial yielding occurs at the upper yield point of 1.3 σ_y although the stress in the most stressed fibers of the beam in the yielded region drops to a value equal to the lower yield point. In Fig. 11, if the author's $\sigma_y + \sigma_y \Delta\sigma'_y$ is replaced by the upper yield point, σ_{yu}, the stress distribution may be visualized as changing from that shown as B to the type shown as A; the yielded zone penetrates into the beam, thereby forcing the understressed fibers to carry more load and thus increasing the resisting moment to the value of the applied moment. The shape of the moment-strain diagram after initial yielding will depend upon the magnitude and distribution of the inherent stress raisers along the most stressed fibers of the beam.

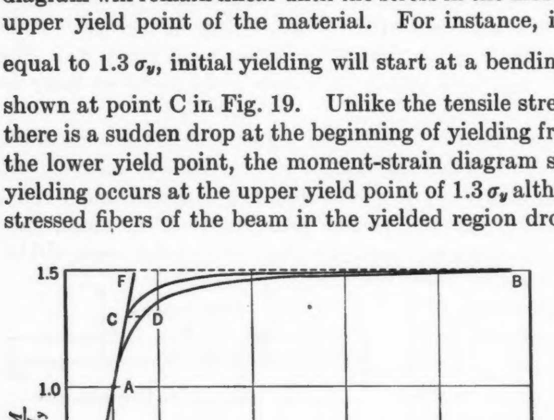

FIG. 19.—MOMENT-STRAIN DIAGRAMS FOR RECTANGULAR BEAMS

The original yielding in a beam usually appears heterogeneously in wedge-shaped zones in regions of localized stress such as at the fillets at point A in Fig. 18. As the load is increased, yielding at a higher load (but at the same stress) will occur in zones B within the test section. Since the strains in the beam are usually measured over a long gage length, localized yielding in the test section may not cause a sudden increase in the measured average strain. Thus, the moment-strain diagram shown in Fig. 19 departs gradually from a straight line at point C. As the applied moment is increased to some value greater than 1.3 M_y, the plastic wedges penetrate to greater depth, thereby increasing the resisting moment; the maximum stress at a section some distance from the plastic region increases correspondingly to a value greater than 1.3 σ_y or until the upper yield point of the material in that part of the beam is reached. The moment-strain diagram for the beam will follow some relationship such as curve OCB in Fig. 19. In any case the moment-strain diagram will be

bounded by the curves OCDB and OFB. The line FB at the bending moment of $\frac{M}{M_y} = 1.5$ is the horizontal asymptote to the theoretical moment-strain diagram and may be considered as the limiting moment for a rectangular beam in which the upper yield point is less than $1.5\ \sigma_y$. This same magnitude for limiting bending moment for a rectangular beam with a definite yield point is also derived by S. Timoshenko (52).

The curves in Figs. 16 and 19 are a development of what the author refers to as the "old theory," as contrasted to the "new theory" attributed to W. Kuntze (which is not strictly a theory but an approximate empirical relationship). The author's discussions of ultimate resisting moment, M_u, in connection with these two theories, seem irrelevant since (as shown by Fig. 16 and the author's test data) initial yielding of a beam occurs (and large plastic deformations are developed) long before the ultimate bending moment is reached. In the tests of Messrs. Morkovin and Sidebottom (see Fig. 17(*a*)), the limiting bending moments closely approximated the value of $1.5\ M_y$ predicted by the "old theory" for a rectangular beam. The data for the author's beam in Fig. 10 also approach a horizontal asymptote at a limiting bending moment approximately 1.5 times the bending moment corresponding to the lower yield point.

The curves in Figs. 16 and 19 also illustrate the effect of the shape of cross section on the magnitude of the maximum upper yield point that may be exhibited without instability in a beam. Initial yielding at an upper yield point of $1.2\ \sigma_y$ in the beam of circular cross section (Fig. 16) probably would not cause a pronounced deviation from linearity in the moment-strain diagram. However, initial yielding at the same upper yield point in the I-beam would cause immediate general yielding throughout the entire depth and a breakdown of the beam, since a bending moment of $1.2\ M_y$ lies above the limiting resisting moment for that shape.

The author's tensile specimen shown in Fig. 8(*a*) would not be adequate to determine the existence of a high upper yield point, although there is some evidence in Fig. 10 that an upper yield point occurred. The writers believe, therefore, that for the moment-strain diagram in Fig. 10 the upper yield point of the material was approximately 20% greater than the lower yield point, and only a value of the lower yield point was obtained from the tensile test. Similarly, in connection with the data shown in Fig. 5 the author states that the Lueders lines "* * * spread quite rapidly down the sides of the specimen * * * with very little increase in the load." This is an indication in itself that an unstable condition existed in which a high upper yield point was developed. Once yielding was initiated in the test section at the upper yield point, the stress on the extreme fiber dropped to the lower yield point and it was necessary for the plastic zone to progress deeply before the resisting moment balanced the unchanged bending moment. Lueders lines would generally develop slowly as loads were increased if the material in the beam did not exhibit an upper yield point.

The appearance of Lueders lines on the surface of a beam is not a sensitive criterion of initial yielding since it involves observation of plastic flow in a considerable part of a member. For beams (particularly those having cross

sections with a minimum of material on the extreme fibers such as the circular and rhombic sections in Fig. 16), the small volume of metal subjected to the peak stress makes it doubtful whether strain markings would become visible until yielding had progressed to an appreciable depth below the surface fibers.

The author's reference to the fatigue test by K. Kloppel (5), in which a beam withstood a large number of cycles of stress equal to the yield point, should not be interpreted as evidence that the yield point is raised in a bend test. No correlation exists between values of endurance limit and yield strength; hence, there would be no reason to suspect that a change in endurance limit would accompany an increase in elastic strength (even if it did occur in a bend test). In fact, in axial tension fatigue test W. L. Collins, Assoc. M. ASCE, and T. J. Dolan (53) found that for five mild steels (subjected to stress cycles varying from zero to a maximum) the endurance limits were about the same as, or somewhat higher than, their yield points. Thus, one would also expect beams made of these same metals to withstand large numbers of cycles of a stress equal to the yield point without evidence of fracture even though some plastic yielding may occur.

In a study of the stresses developed around a notch in a beam subjected to pure bending (38), X-ray diffraction patterns were used to determine the stress at which yielding was initiated. In these tests a sharp stress gradient was present—resulting in a stress concentration factor of about $2\frac{1}{2}$ at the notch; any effects of delayed yielding resulting in higher elastic stresses should be even more pronounced under these conditions than in the tests reported by the author. However, Mr. Norton concludes that:

> "* * * the stress concentration and the stress gradient resulting from bending have no effect on the initiation of plastic flow in the immediate vicinity of the notch. There the state of stress is uni-axial and plastic flow sets in at the yield stress for pure tension."

For many applications of beams made of ductile steel, considerable yielding of the outer fibers can occur without causing excessive curvature or deflection of the member; hence, for some uses the investigators may infer that the beam is not structurally damaged as a whole. Certain beams may resist loads greater than those required to start yielding in the extreme fibers. It should be recognized that the increased strength is not the result of "higher elastic stresses" but rather that, because of the redistribution of stress, the understressed part can offer greater resistance to the load without permitting sufficient plastic distortion to the member as a whole, to constitute appreciable structural damage. The curves in Fig. 16 indicate the relative extent to which beams of various shapes may be overloaded (stressed beyond the yield point) before appreciable damage is apparent by their relative deviations from a linear load-strain relationship. The plastic deformations of the metal in the extreme fibers corresponding to these small deviations from linearity are rather large, however, as indicated roughly by the percentage by which the ratio ϵ/ϵ_y exceeds unity. This fact might require serious consideration in certain applications where danger of instability because of localized buckling may be aggravated by the plastic deformation of the most stressed fibers.

In conclusion, the basic behavior of flexural members as visualized by the writers may be summarized as follows:

(*a*) The materials that do not exhibit an upper yield point, the actual stress necessary to cause initial yielding of the most stressed fibers of a member subjected to a nonuniform stress distribution (such as that which occurs in a beam) is the same as that required to cause yielding under a uniform stress as obtained in a standard tensile test specimen.

(*b*) For materials exhibiting an upper yield point, the stress gradient that occurs in a beam makes it possible to obtain, before yielding occurs, a computed stress somewhat larger than the yield point that is ordinarily determined as the average stress in a tension specimen. This is true since the magnitude of the upper yield point as computed from the tension test is markedly decreased by unfavorable test conditions, whereas these test conditions do not have as pronounced an effect in decreasing the apparent upper yield point in a beam.

(*c*) The nonuniform distribution of stress that occurs in a beam enables the member to resist static loads considerably greater than those required to start yielding of the extreme fibers. As indicated in Fig. 16, the shape of the cross section of the beam will determine the extent to which plastic deformation of the extreme fibers may occur before structural damage is evidenced by appreciable distortion of the member as a whole.

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DISCUSSIONS

FACTORS CONTROLLING THE LOCATION OF VARIOUS TYPES OF INDUSTRY

Discussion

By Harold M. Lewis, and R. F. Goudey

Harold M. Lewis,[4] M. ASCE.[4a]—The part of the civil engineer in the location of industry has been, primarily: Supplying some of the accessories to a successful site, such as transportation facilities; laying out site utilities, such as water supply, drainage, streets, and terminals; and designing foundations and other structural elements of the plant itself. The fact that this paper was presented at a joint meeting of the City Planning Division of the American Society of Civil Engineers and the American Institute of Planners indicates that civil engineers are taking an active interest in some of the broader phases of the problem. Instead of simply having an industrial site handed to them, they should play an increasing rôle in helping their clients determine where that site should be.

Mr. Wood has indicated that the location of industry has followed certain laws based on accessibility to raw materials, convenience for manufacturing (power and labor), and accessibility to markets. These essentially physical factors are being supplemented by what might be called social and psychological factors. Sites which might not have been considered under the old standards may become desirable because the workers (labor) find the locality a pleasant place to live in and one which affords higher standards of housing, health, recreation, and education. The owners and executives (business) may find that better municipal government and sound city planning provide savings which more than counterbalance some disadvantages in accessibility of raw materials, power, and markets. In other words, a community that is well planned and administered as a result of good city and regional planning may find certain industries coming to it primarily because of those characteristics.

There will always be certain industries that are naturally better adapted to central urban areas, others that tend to gravitate to suburban areas adjoining large cities, and still others that require enough space and employ a sufficient

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[4a] Received July 9, 1946.

variety of workers to enable them to go out into new areas and establish communities of their own. The economic and industrial survey undertaken by the Committee on a Regional Plan of New York and Its Environs,[5] classifying the industries in the region along these lines, was a pioneer study of its kind.

In the first, or urban, group were those small-scale industries with a seasonal labor force and therefore a high turnover of employees, or in which the time factor in contacting the consumer or purchaser was important. The group included certain types of men's and women's clothing, high-grade jewelry, photo-engraving, job printing, cosmetics, and perishable foods. In the second, or surburban, group were many of the heavy metals and chemical industries, refineries, printing of periodicals, bookbinding, textile finishing, lumber mills, and women's underwear. In the third group the writer would place large-scale munitions manufacture, mining, and the assembly and testing of airplanes. Among the cities that have been built up, or greatly expanded, as a result of the establishment of new industries on new sites are: The mining cities of the west such as Leadville, Colo.; the steel cities such as Gary, Ind.; the automobile cities such as River Rouge, Mich.; the rubber cities such as Akron, Ohio; and the iron and steel cities such as Birmingham, Ala.

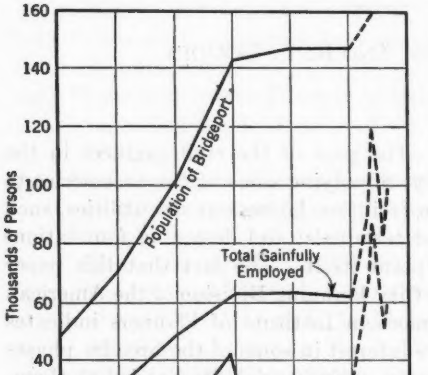

FIG. 2—COMPARATIVE TRENDS IN POPULATION AND WAGE EARNERS IN BRIDGEPORT, CONN., 1899–1946

Mr. Wood referred to the present postwar readjustment period. The writer recently had occasion to study industrial trends in Bridgeport, Conn.—a fairly typical manufacturing center in the 100,000 to 500,000 population range. Like many other cities, it is going through such a period; but, again like many other cities, it had a somewhat similar experience following World War I. The curves in Fig. 2 indicate the similarity and the differences in these two experiences. The manufacturing boom of World War I showed a peak in 1919 in the number of manufacturing wage earners. Then came a sudden drop, with relatively minor changes up to 1939.

During the succeeding four years, all four curves showed second wartime jumps, the outstanding feature being the increase in the total number gainfully employed from 62,266 in 1940 to a peak of 119,500 in the fall of 1943, paralleled by similar increases in the number of manufacturing wage earners. The

[5] "Major Economic Factors in Metropolitan Growth and Arrangement," Regional Survey of New York and Its Environs, Vol. I, 1927, pp. 19–30, and 104–107

inevitable downward trend came in 1944, but the City of Bridgeport had anticipated this trend. Through its city administration, the Chamber of Commerce, and a representative planning council organized by the Chamber of Commerce in 1943, the problems of both the city and region of which Bridgeport forms a center had been attacked by eleven committees. It was found that, although the city area affords few sites for new large-scale industries, there are ample sites for both industry and port development within the region. These were mapped; their relative advantages and disadvantages were analyzed; and harbor improvements to make them more accessible were proposed.

It is expected that, whereas postwar trends will result in more production per employee, new manufacturing buildings will provide greater floor area per worker, more light and air within the buildings, and more open ground about them. The open space should be used to provide both automobile parking and recreation facilities for employees.

In 1946 a recovery from the low point of 1945 had already occurred and the Bridgeport Chamber of Commerce stated:

> "With the experience of World War I to look back on the industrial leaders and business men anticipated a mass exodus of the huge tide of in-migrant workers that flooded this area to reap the harvest of high war-time wages. This exodus did not take place, only a trickle left. * * * With the easing of parts, supplies and raw materials, Bridgeport is looking forward to its greatest peace-time production records."

In this case, advance planning seems likely to stabilize employment at a level considerably above that of the prewar period.

R. F. Goudey,[6] M. ASCE.[6a]—That industries should be located on the basis of sound engineering evaluations of source of raw materials, availability of labor, location of markets, transportation facilities, and availability of power has been emphasized by Mr. Wood. Rarely are such studies made and it is more common to have industries located by representatives of a local chamber of commerce and by public officials not familiar with the critical problems involved.

In too many instances, industries locate in new territory through decisions based largely on misinformation and even ignorance. Usually when an eastern industry is about to move westward, someone in its organization who is ready for a vacation takes a trip west to make a casual survey; or a written inquiry is forwarded to a local chamber of commerce or a business division of a power utility. In one large metropolitan area, the representative of a prospective industry, if routed through chamber of commerce channels, might have to meet fourteen public officials of the municipality before obtaining correct information on such items as taxes, cost of water, facilities for sewage and waste disposal, power cost, housing facilities, and availability of labor.

On the other hand, if the inquirer entered the area through contact with a large private power utility, he might be induced to locate in the county area instead of the city, in which case he would have to interview twelve or more county officials to obtain information on the points involved in the selection

[6] San. Engr., Dept. of Water and Power, Los Angeles, Calif.

[6a] Received August 5, 1946.

of a new site. In any event, he is kept ignorant of the advantages which other near-by areas might have over the one he first investigated. In many cases the preliminary stages have been passed and a final decision has been made to locate the plant at a given place when, for the first time, engineers for the industry learn that the quality of water is not satisfactory for their purposes, or that they are unable to dispose of liquid waste without polluting the underground water supply, or that the cost of water as indicated by the metered rate schedule did not disclose that there was an additional district tax of 50¢ per $1,000 valuation, or that adequate protection from floods could not be obtained except at exorbitant cost. Such information was not obtained in the preliminary stages because there was no proper coordination of engineering information from the different departments involved in the problem.

Cheap water is essential for cooling water, heat exchange purposes, processing, and fire protection for rayon plants, rubber products plants, woolen mills, paper pulp plants, and tanneries. In cities where domestic water must be imported from long distances, it may be desirable to reclaim sewage for a cheap water supply of ample volume to serve plants having large water requirements. Interesting instances where sewage has been conserved for commercial purposes are as follows:

1. El Tovar, Grand Canyon, Arizona, has reclaimed sewage for boiler, toilet flushing, and engine use since 1925;
2. The Bethlehem Steel Company at Baltimore, Md., uses treated city sewage for cooling water;
3. The Barnsdall Oil Company at Corpus Christi, Tex., uses reclaimed sewage for cooling purposes;
4. The Shell Oil Company at Signal Hill, Calif., at one time used all the treated Signal Hill sewage for industrial purposes;
5. The Kaiser Steel Mill at Fontana, Calif., re-uses all its wastes so that there is little actual waste;
6. In Pasadena, Calif., treated sewage is discharged into the Rio Hondo and is the only supply for certain irrigation ditches in the Riviera District; and
7. In Fresno, Calif., sewage is discharged into aquifers from which irrigation water is developed.

These cases show clearly that, where the original cost of water is high or where it must be transported long distances or be developed at a high cost, reclamation is not only practical but economical.

Many industries attempt to relocate because of trouble experienced with the matter of waste disposal at their old location. Industries have a habit of trying to make little of their waste disposal problems and so make it difficult for consulting engineers to advise them as to a proper location. Although many industries have spent considerable sums in attending to their liquid waste problems, the amount spent by communities has been much greater. There has been considerable duplication of expense. Therefore, these matters should be given consideration during the period when the original location of industries is being considered. In California, the following experience with industrial waste has been particularly troublesome:

(*a*) In twenty cities there has been trouble with peach waste where lye wastes, spread on the land, resulted in mineral pollution of ground waters;

(*b*) Fourteen cities have had trouble with tomato waste in which lye waste has been discharged on to the land;

(*c*) Fourteen cities have had trouble with salt and lye waste from olive packing plants and these wastes have been placed in underground water supplies;

(*d*) Twelve cities have had trouble with creamery waste, particularly where whey was involved;

(*e*) Six cities have had trouble with packing pimientos and strong alkali waste with garbage has been placed on to the land;

(*f*) Five cities have had trouble with slaughterhouse wastes and have had to install grease removal facilities;

(*g*) Five cities have had difficulty with refineries and separate disposal was necessitated;

(*h*) Five cities have had trouble with winery waste which is now disposed of by land;

(*i*) Four cities have had problems with sugar plant waste, two having put in separate systems and two having ceased operation; and

(*j*) Three cities have had trouble with fish waste and in all instances special works have had to be installed.

The following are typical of the many special problems involved in practically every type of industry:

Anaheim, Calif.—The United States Industrial Alcohol Company discharged molasses waste in the Orange County Outfall Sewer, causing such a terrific production of hydrogen sulfide that the company was forced to disconnect from the sewer and install local treatment works. This situation resulted, however, in pollution of underground water supplies with mineral constituents.

Betteravia, Calif.—Betteravia has its own waste disposal system independent of any municipal system.

Corona, Calif.—The By-Products Exchange Plant at Corona discharged wastes into the Corona Sewer System, resulting in the pollution of several private wells. The company installed a separate system in cooperation with the City of Corona for adequate treatment and proper disposal. There is still some pollution of underground supplies, but no wells so far have been affected.

Guadalupe, Calif.—The creameries in Guadalupe installed a sewer system at their own expense to dispose of the industrial waste without affecting any underground water supplies.

Lemoore, Calif.—The city constructed a biofiltration plant consisting of aeration and with recirculation and a rock filter based on a year's experiment with creamery wastes; but, when the plant was put in operation, it proved to be a complete failure because of the acid lactose splitting organisms which thrived in the aeration tank and its recirculated sludge. Separate disposal had to be provided.

Los Angeles, Calif.—Los Angeles has had trouble with the disposal of alkali, salt brine, chromium, phenol, and solvents which have reached certain strata of local underground water supplies.

Ontario, Calif.—The Citrus By-Products Plant at Ontario installed a separate sewer to seepage beds on land, resulting in the pollution of near-by wells. Suit was brought against the company which was settled outside court; but the city extended City of Ontario water to these water consumers. In the meantime, however, there is continued pollution of the underground water supply.

Santa Ana, Calif.—Two sugar plants of the Hollywood Sugar Company and of the Santa Ana Sugar Company used to have a sewer system carrying the combined wastes to the ocean. These plants discontinued operation for several reasons—one of which was that their sewer facilities were no longer available.

Tulare, Calif.—Tulare built an activated sludge plant with the idea of treating domestic sewage. The chamber of commerce invited a creamery which produced whey waste to Tulare. The volume of waste amounted to 245,000 gal per day and gave an equivalent load of 80,000 people. This demand was about ten times what the treatment plant could meet and the plant became a complete failure. Separate disposal had to be provided.

Summary.—It is evident that the question of waste disposal should be given far more attention than has been the case in the past. It would greatly assist managers who are considering the problem of relocating industries to deal with a coordinating committee representing all sources of information that management should have in arriving at a final decision as to where industries should finally be located. In some metropolitan areas this committee might require representatives from public and private utilities as well as the heads of city and county departments supplying services to industries. It is strange that large industries appear to avoid the services of consulting engineers in deciding on new locations. The problem is sufficiently involved to warrant such consultation.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

CRITICAL STRESSES IN A CIRCULAR RING

Discussion

BY E. P. POPOV

E. P. POPOV,[19] JUN. ASCE.[19a]—The problem of determining the critical stresses in a circular ring, subjected to two concentrated opposing radial forces on the outer boundary, is not a new one. The solution is of practical importance in certain bearing and machine design problems as well as in the case cited by Messrs. Ripperger and Davids.

Since the series used by the authors to obtain a solution is shown to be nonconvergent in expressing the radial boundary stresses under the concentrated loads, the procedure may appear questionable. The use of this nonconvergent series can be avoided entirely by a method first suggested by S. Timoshenko.[4, 20] Such a method was further recommended by L. N. G. Filon.[7] The Timoshenko method utilizes a known finite solution for two equal and opposite concentrated radial forces applied on the outer boundary of a solid cylinder. Then a concentric hole, equal to the bore of the hollow cylinder, is cut out of such a solid cylinder. Normal and shearing forces exist as distributed forces around the edge of the hole to maintain the state of stress in the remaining hollow cylinder equivalent to that in the original solid one. These edge forces can be expressed by convergent series. Application of equal and opposite forces around the edge of the hole can then be used for the determination of the constants of integration in the stress function[21] in terms of these boundary stresses. The resulting stresses are superposed on the stresses obtained from the finite solution of a solid cylinder to obtain true stress in a hollow cylinder. Displacements can be treated in a similar manner.

NOTE.—This paper by E. A. Ripperger and N. Davids was published in February, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1946, by Robert H. Philippe and Frank M. Mellinger.

[19] Asst. Prof., Civ. Eng., Univ. of California, Berkeley, Calif.

[19a] Received August 5, 1946.

[4] "On the Distribution of Stresses in a Circular Ring Compressed by Two Forces Acting Along a Diameter," by S. Timoshenko, *The London, Edinburgh and Dublin Philosophical Magazine and Journal of Science*, Vol. 44, No. 259, July, 1922, p. 1014.

[20] "Stresses in a Ring Compressed by Two Forces Acting Along a Diameter," by S. Timoshenko, *Bulletin*, Kiev Polytechnic Inst., 1910 (in Russian).

[7] "The Stresses in a Circular Ring," by L. N. G. Filon, *Selected Papers*, Inst. C. E., London, 1924.

[21] "On the Direct Determination of Stress in an Elastic Solid, with Application to the Theory of Plates," by J. H. Michell, *Proceedings*, London Mathematical Soc., Vol. 31, 1899, p. 111.

The special case of this problem by the foregoing method for $\bar{r} = 0.5$, as noted, was solved by Professor Timoshenko,[4, 20] and extended further by his students. The case of $\bar{r} = \frac{1}{3}$ was solved by V. Billevicz,[22] and a general solution for any value of $\bar{r}$ was obtained by J. Maulbetsch, as reported by C. W. Nelson.[23]

The foregoing method involves the use of certain series. The radial, tangential, and shear stresses for a solid circular cylinder in the series form are shown[23] to be:

$$\sigma_r = \frac{2\,P}{\pi\,r_o}\left[-\tfrac{1}{2} + \cos 2\,\theta - \left(2\,\frac{r^2}{r_o^2} - \frac{r^4}{r_o^4}\right)\cos 4\,\theta + \left(3\,\frac{r^4}{r_o^4} - 2\,\frac{r^6}{r_o^6}\right)\cos 6\,\theta\cdots\right] \quad (25a)$$

$$\sigma_\theta = \frac{2\,P}{\pi\,r_o}\left[-\tfrac{1}{2} + \left(2\,\frac{r^2}{r_o^2} - 1\right)\cos 2\,\theta - \left(3\,\frac{r^4}{r_o^4} - 2\,\frac{r^2}{r_o^2}\right)\cos 4\,\theta\cdots\right] \quad (25b)$$

and

$$\tau_{r\theta} = -\frac{2\,P}{\pi\,r_o}\left[\left(1 - \frac{r^2}{r_o^2}\right)\sin 2\,\theta - 2\left(\frac{r^2}{r_o^2} - \frac{r^4}{r_o^4}\right)\sin 4\,\theta + 3\left(\frac{r^4}{r_o^4} - \frac{r^6}{r_o^6}\right)\sin 6\,\theta\cdots\right] \quad (25c)$$

These series are convergent and with a desired value of radius r can be used as the boundary stresses at the bore. In these series the angle θ is measured from a horizontal axis to the right of the origin. The loads are applied vertically.

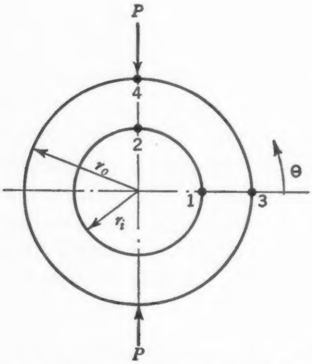

FIG. 9.—THE LOADED CIRCULAR CYLINDER

For the derivation of Eqs. 25, and of other formulas pertaining to the analytical solution of the hollow circular cylinder, the readers are referred to Mr. Nelson's dissertation,[23] in which the problem is fully treated. However, for the purposes of comparison with the paper by Messrs. Ripperger and Davids, certain critical values compiled by Mr. Nelson are quoted. Perhaps the most significant points of stress on the hollow cylinder are those marked 1, 2, 3, and 4 in Fig. 9. The stress concentration factors for the tangential stresses corresponding to these points are shown in Table 4 for the different diameter ratios. The factors of stress concentration for the maximum tensile stresses are given through and beyond the di-

[22] "Analysis of Stresses in Circular Rings," by V. Billevicz, thesis presented to the Univ. of Michigan at Ann Arbor in 1931 in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

[23] "Stresses and Displacements in a Hollow Circular Cylinder," by C. W. Nelson, thesis presented to the Univ. of Michigan at Ann Arbor in 1939 in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

ameter ratios presented by the authors in Table 1. Table 4 also shows some other values for additional points that may be of interest. The comparable points of stress are in complete agreement with the results obtained by Messrs. Ripperger and Davids, which is reassuring. In connection with Table 4, it may be worthy to note that, in these elastic analyses, only small deformations are considered, and thus a solution for a case such as $\bar{r} = 0.9$ should be questioned.

The solution devoelped by the authors, as well as the one used for obtaining the results shown herein, treats the problem of a hollow cylinder as a two-dimensional problem. In practice, including the case of foundation rock samples, the cylinder is of finite length, compressed between the flat surfaces of two larger bodies. The latter is not a two-dimensional problem. Actually the load distribution along the length of the load is not uniform.

TABLE 4.—CONCENTRATION FACTOR K

$$\left(\sigma_\theta = \frac{P\,K}{\pi\,r_o}\right)$$

$\bar{r} = \frac{r_i}{r_o}$	Points (See Fig. 9)			
	1	2	3	4
0.9	353.80	−581.59	−299.42	512.22
0.8	92.07	−140.72	− 64.65	106.73
0.7	42.99	− 60.45	− 24.42	38.25
0.6	25.720	− 32.838	− 11.411	16.536
0.5	17.798	− 20.311	− 5.857	7.529
0.4	13.702	− 13.682	− 3.063	3.214
1/3	12.121	− 10.917	− 1.931	1.573
0.3	11.563	− 9.856	− 1.503	0.976
0.2	10.531	− 7.598	− 0.607	−0.218
0.1	10.108	− 6.385	− 0.143	−0.816

Theoretically, considering the pressures between two bodies in contact, Mr. Nelson[23] shows that the load distribution along the length of the cylinder is approximately uniform except near the ends of the areas of contact, provided the width of actual contact is small compared to the length of the cylinder.[24] Since such requirements are usually met, these solutions are adequate.

TABLE 5.—COMPARISON OF STRESSES FOR POINT 2 IN FIG. 9

Diameters, in Inches		Ratio of Col. 1 / Col. 2	Load P (lb per in. of length)	Tangential Stress (Lb per Sq In.)		Ratio of Col. 5 / Col. 6
$2\,r_i$	$2\,r_o$			Photoelastic	Computed	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0.496	2.374	0.104	892	1,554	1,530	1.015
0.496	2.374	0.104	1,135	1,954	1,950	1.00
0.500	2.374	0.210	515	1,080	1,100	0.99
0.505	2.374	0.212	892	1,820	1,870	0.97
0.505	2.374	0.212	1,135	2,330	2,380	0.98
0.752	2.375	0.316	515	1,465	1,435	1.02
0.752	2.375	0.316	892	2,510	2,490	1.01
0.752	2.375	0.316	1,135	3,220	3,170	1.01
1.465	2.375	0.617	204	1,940	1,905	1.02
1.465	2.375	0.617	354	3,430	3,305	1.04
1.465	2.375	0.617	515	4,830	4,800	1.01

[24] "Elastische Beruehrung zweier Halbraeume," by G. Lundberg, *Forschung auf dem Gebiete des Ingenieurwesens*, September-October, 1939, pp. 201–211.

The theoretical results are further verified by photoelastic tests briefly reported by O. J. Horger.[25] More details pertaining to these tests may be of interest and some are cited,[23] for comparison, in Table 5. The agreement between the theoretical and experimental results is seen to be excellent.

The writer is happy to offer these comments from a source not generally available as a verification and extension of the results presented by the authors.

[25] "Photoelastic Analysis Practically Applied to Design Problems," by O. J. Horger, *Journal of Applied Physics*, July, 1938, pp. 457–465.

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DISCUSSIONS

NEW PROJECT FOR STABILIZING AND DEEPENING LOWER MISSISSIPPI RIVER

Discussion

By Gerard H. Matthes

Gerard H. Matthes,[4] Hon. M. ASCE[4a]—The author is to be commended for the conciseness with which he has presented the historical and physical sequences of flood control and channel regulation in the Lower Mississippi Valley; also, for setting forth so clearly the stable characteristics of the river as regards its being neither an aggrading nor a degrading stream. This truly remarkable stability, despite its ceaseless meandering, has been fundamental to the success of much of the engineering work executed. Broadly viewed, it is the basic justification for the large expenditures for flood control and river-channel improvement that have been made in the past, and for the expenditures yet to be made.

Had the physical conditions in the Lower Mississippi Valley paralleled those in Mesopotamia (Iraq), its history would have been differently written. Mesopotamia, an alluvial valley of comparable size, has been, and still is, outstanding for the difficulties that beset its large irrigation and flood control works. The sediment deposited by two large rivers, the Tigris and the Euphrates, raises river beds and flood plain at a rapid rate. Great floods, repeatedly, have buried, under silt, the extensive irrigation systems, levees, and even entire cities whose ruins have been found at successive levels underground. The alluvium, furthermore, has built out into the Persian Gulf 170 miles in 5,500 years or at the average rate of 160 ft a year. In marked contrast are the extremely slow advance of the mouths of the Mississippi into the Gulf of Mexico, and the absence of bed aggrading in the 1,000 miles of Lower Mississippi River.

The river has suffered aggradation only to the extent of a few feet at its junction with the Ohio due to local conditions downstream created by the hand of man. In other places, the bed has become lowered as much as 5 ft where cutoffs or dredging operations have shortened the channel. Aside from these

Note.—This paper by Charles Senour was published in February, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1946, by Charles W. Okey, and F. Newhouse.

[4] New York N. Y.

[4a] Received October 3, 1946.

local changes (which resulted from channel improvement), the low-water profile has shown but little change. An interesting checkup became available in 1936 when the low-water profile of that year was compared with that of 1901, both representing extreme low-water conditions with fairly comparable discharges. It was found that the elevation above sea level, of the water surfaces at New Madrid, Mo., Memphis, Tenn., Vicksburg, Miss., and Angola, La., on these two occasions were in close agreement, differences ranging from 1 ft to almost zero. The important point brought out by this comparison was that, wherever the river had not become affected by engineering operations, its low-water profile had not been altered by natural causes over a period of 35 years.

These stable characteristics of the Lower Mississippi are referred to by the author when he terms it a "poised river." As this is an unfamiliar term as applied to rivers, a word of explanation is in order. The word "poised" was first used in 1941[5] to denote a status in which a stream shows no apparent natural tendency toward over-all raising or lowering of its bed. It is merely a convenient label for such a status in connection with the construction and maintenance of engineering works, the useful life of which commonly does not exceed half a century. The word implies no permanency, however; rather, it implies the possibility that the status may change at any time, or may be in the process of changing, as measured over the long course of geologic time. A river may be "poised" regardless of whether, concurrently, the land surface of its flood plain is being raised or eroded. Such, in substance, was the prevailing status of the Lower Mississippi up to 1932 when major channel improvements were undertaken.

The subject of the river's stability is of interest to engineers for another reason; namely, the widespread misunderstandings that are entertained concerning it. Statements have emanated from supposedly well-informed sources[6,7,8,9] to the effect that the Lower Mississippi is "forever" building up its bed and banks to higher levels, and that, consequently, the levees built for protection against overflow will have to be raised "indefinitely" as flood crests reach higher elevations. In a geological sense, the river is actually an aggrading river, because were it not for the levees it would continue to raise its flood plain by depositing silt. Deposits of this kind still take place on overflowed areas which have no levee protection, as evidenced by the gradual filling of lakes and sloughs in backwater areas. On the whole, however, the silt layers deposited by the Lower Mississippi, as described by the author, have been much thinner than commonly imagined; otherwise they would have obliterated the early channel courses of the river of 2,000 years ago.[10]

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[5] "Basic Aspects of Stream Meanders," by Gerard H. Matthes, *Transactions*, Am. Geophysical Union, 1941, p. 632.

[6] *Compton's Pictured Encyclopedia*, 1940 Ed., Vol. 9, p. 204; and Vol. 12, p. 110.

[7] *World's Work*, May, 1913, p. 23.

[8] *National Geographic Magazine*, July, 1926, p. 118.

[9] *Harpers Monthly Magazine*, September, 1936, p. 376.

[10] Report on "Geological Investigation of the Alluvial Valley of the Mississippi River," by Harold N. Fisk, Mississippi River Commission, Vicksburg, Miss., 1945.

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DISCUSSIONS

ANALYSIS OF UNSYMMETRICAL BEAMS BY THE METHOD OF SEGMENTS

Discussion

BY VICTOR R. BERGMAN, AND A. A. EREMIN

VICTOR R. BERGMAN,[10] ASSOC. M. ASCE.[10a]—In the analysis of beams of variable moments of inertia by the moment-distribution method, the work preliminary to the actual distribution frequently requires a major part of the total time involved. The determination of fixed-end moments, stiffness factors, and carry-over factors can be a very tedious and time-consuming task, unless facilitated by the use of tables and diagrams of adequate scope. Therefore, the author's presentation of a fairly simple method for extending the range of applicability of readily available data is to be commended. Incidentally, Eqs. 5 are not peculiar to the problem under discussion; they have been developed elsewhere in connection with other phases of structural analysis.[11]

Several years ago, the writer had occasion to develop a "Method of Sections" in which the procedure for the calculation of fixed-end moments is practically identical with that presented by Mr. Lifsitz, but in which the method of computing stiffness factors and carry-over factors is somewhat simpler.

Consider the unsymmetrically haunched beam shown in Fig. 7 for which the author, by use of Fig. 8, determined values of K and C for each end of both segments. The writer's procedure for determining values of K_{AB} and C_{AB} involves four simple steps, as follows:

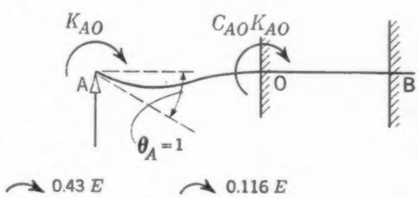

FIG. 13.—STEP 1

Step 1.—Assume the beam to be both supported and fixed against rotation at point O, and hinged at point A. Apply to end A (Fig. 13) a clockwise moment equal to K_{AO} ($= 0.43\ E$).

NOTE.—This paper by Sol Lifsitz was published in March, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1946, by William A. Conwell, Ralph W. Hutchinson, and Thomas P. Revelise.

[10] Vice-Pres., Godwin Constr. Co., New York, N. Y.

[10a] Received August 26, 1946.

[11] "One-Story Concrete Frames Analyzed by Moment Distribution," *Concrete Information Bulletin No. ST42*, Portland Cement Assn.

This will produce on end O of the segment AO a clockwise moment equal to $C_{AO} K_{AO}$ ($= 0.116\ E$).

Step 2.—At end A, the applied moment will have produced (by the definition of stiffness factor) a unit rotation. Lock end A in this rotated position and unlock joint O, distributing moments as shown in Fig. 14.

Step 3.—Determine the reaction provided by the temporary support at point O, as follows:

$$F'_O = \frac{0.360\ E + 0.061\ E}{20} + \frac{0.061\ E + 0.056\ E}{15} = 0.0289\ E \ldots\ldots (25)$$

Next remove the temporary support at point O, and replace it with an equal but opposite load. In the computation preceding Eqs. 20, the author found

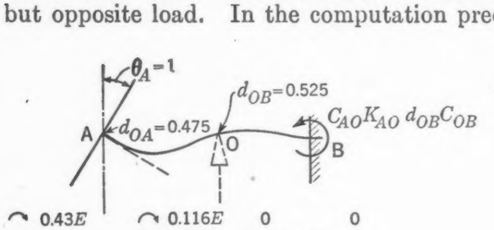

FIG. 14

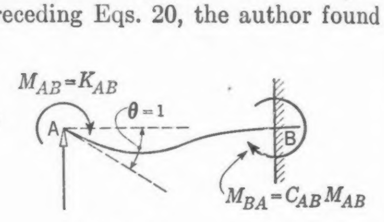

FIG. 15

that a load of 4.30 E at point O corresponds to moments at ends A and B equal, respectively, to:

$$m_{AO} = 28.75\ E;\ \text{and}\ m_{BO} = 22.97\ E \ldots\ldots (26)$$

By proportion, for a load of 0.0289 E at point O,

$$m_{AO} = \frac{0.0289\ E}{4.30\ E} \times 28.75\ E = 0.193\ E \ldots\ldots (27a)$$

and

$$m_{BO} = \frac{0.0289\ E}{4.30\ E} \times 22.97\ E = 0.154\ E \ldots\ldots (27b)$$

Step 4.—Combine the end moments calculated in step 3 with those determined in step 2, to arrive at the final end moments at A and B (Fig. 15). After receiving a unit rotation, end A has been maintained in that position; it is evident, therefore, that the final moment ($= 0.167\ E$) is equal to K_{AB}, the required stiffness factor, in accordance with the definition of the term. Furthermore, by the definition of carry-over factor, $C_{AB} = \frac{M_{BA}}{M_{AB}} = \frac{0.098\ E}{0.167\ E} = 0.586.$

The values thus found are seen to check, very closely, those calculated by the author. The stiffness and carry-over factors at end B, of course, can be determined in a manner similar to that followed at end A. The foregoing method involves only basic moment-distribution concepts, and, therefore, it has some slight advantage in ease and simplicity over that proposed by the author.

A. A. EREMIN,[12] Assoc. M. ASCE.[12a]—A temporary support, inserted hypothetically, in an unsymmetrical member simplifies the computation of moments, stiffness, and carry-over factors. The algebraic procedures used by Mr. Lifsitz for this purpose can be clarified by the use of graphics. For illustration, the beam in Fig. 1 is analyzed graphically in Fig. 16. The member is sustaining a unit load at the midpoint of distance AO in Fig. 16(*b*).

The characteristic points in the member with a temporary support at point O are shown by the three-line polygon, Fig. 16(*a*). The moments in the member with the temporary support, fixed against linear displacement, are determined by constructing cross lines as shown in Fig. 16(*b*). The effect of a unit displacement at the temporary support is shown in Fig. 16(*c*). The final moments in the member are obtained by the summation of moments in Fig. 16(*b*) and Fig. 16(*c*), multiplied by the factor of elastic displacement of the support at point O. The resulting moments check the values in Fig. 10 very closely. Elsewhere[13] the writer has demonstrated a similar graphical construction for the distribution of moments in a continuous beam with a suspension hinge.

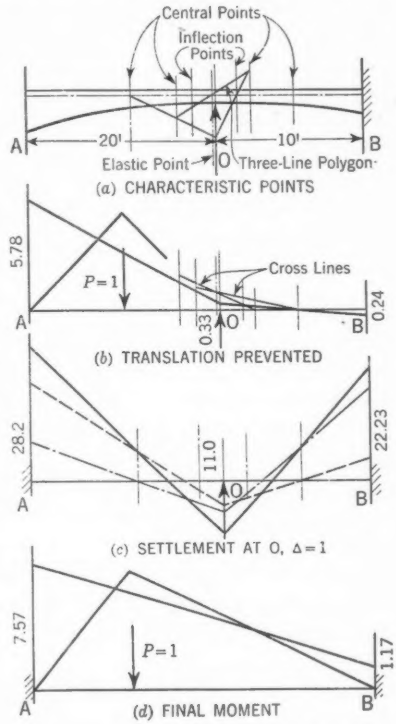

FIG. 16.—GRAPHIC CONSTRUCTION OF MOMENTS IN AN UNSYMMETRICAL BEAM

The author's algebraic formulas for moments, stiffness, and carry-over factors serve as the preliminary steps in the distribution of moments in continuous beams. The graphical principles in Fig. 16 may be applied directly to the distribution of moments in any continuous beam or frame containing unsymmetrical members.

Unsymmetrical members are often used in modern engineering structures; therefore, the paper constitutes a valuable contribution in the analytical interpretation of the temporary support method.

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[12] Associate Bridge Engr., California State Highways Dept., Sacramento, Calif.

[12a] Received September 6, 1946.

[13] "Analysis of Continuous Frames by Graphical Distribution of Moments," by A. A. Eremin, Sacramento, Calif., 1943.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

DESIGN OF PLYWOOD I-BEAMS

Discussion

BY DICK W. EBELING, I. OESTERBLOM, AND C. J. HOGUE

DICK W. EBELING,[8] JUN. ASCE.[8a]—There are a few points in this very useful contribution which are not entirely clear: First, the author states (see "Introduction") that the unit horizontal shear is eight times the web-flange shear; but he fails to state that the unit horizontal shear in the plywood is twice the allowable horizontal shear for the wood from which the plywood is made. Second, from Fig. 1, it would seem that the flange depths must be equal, which is not true. The form factor depends only on the depth of the compression flange. The Forest Products Laboratory in Madison, Wis., has developed a formula[9] for the most efficient section with unbalanced flanges. In using this formula, the section is first designed with equal flanges; then part of the tension flange is transferred to the compression flange keeping the total area, height, and width constant:

$$x = \frac{A\, b_f\, d^2 - \sqrt{A^2\, b^2_f\, d^4 - 4\, A\, I_s\, b_f\, d(b_f - b_w)\, d_w}}{2\,(b_f - b_w)\, d_w\, b_f\, d} \qquad (13)$$

in which x is the thickness to be transferred from the tension to the compression side; and I_s is the moment of inertia of the symmetrical section.

Also, the spacing of stiffeners given in Fig. 4 is actually the minimum spacing. Just as the stirrup spacing in a concrete beam increases as the shear decreases, the stiffener spacing can be increased as the shear decreases. However, it is not possible to increase the spacing beyond 5 d_w because the stiffeners are needed as flange spacers in fabrication.

To increase the strength in shear and to decrease the deflection due to shear, the plywood web may be placed with the face grain at an angle of $\pm$ 45° to the span—thus doubling the allowable horizontal shear in the web. However, the strength in bending is reduced to one fourth of the value for plywood parallel to the span, although the allowable web-flange shear is not

NOTE.—This paper by Howard J. Hansen was published in June, 1946, *Proceedings*.

[8] Asst. Engr., U. S. Army Engrs., Portland Dist., Powerhouse Development Branch, Portland, Ore.

[8a] Received July 26, 1946.

[9] "ANC Handbook on Design of Wood Aircraft Structures." U. S. Govt. Printing Office, Washington, D. C., 1943, Supplement No. 2, p. 3.

affected. The modulus of rigidity of the plywood is increased five times, which reduces the deflection.

If the plywood is placed with the grain at an angle to the span, the beam will probably have to be fabricated completely in the shop. In most cases, it is better to have the beams completely fabricated in the shop, where the trained men and proper facilities are available. Also, in shop fabrication, the splice plates can be eliminated by scarfed splice joints which develop the full strength of the plywood.

The author's theoretical method of spacing stiffeners agrees closely with that recommended by the Douglas Fir Plywood Association[2] as checked by tests.[10] Also a number of planes were designed from the Forest Products Laboratory recommendations[11] which are similar to the author's method; and, as far as the writer knows, there were no structural failures of the plywood I-beams and box beams. Therefore, the author's method is adequate and conforms to conventional factors of safety for wooden structures.

I. OESTERBLOM,[12] M. ASCE.[12a]—To see papers on plywood construction is refreshing in a double sense. It reminds one of the not so far away past when rolled iron sections were "thrown" at an unsuspecting world; a keen search then followed to discover all their potential uses. It also affords a view of a future in which the United States will certainly go abroad begging for the iron ores she needs to maintain industrial supremacy, having tried valiantly—almost to fanaticism—to exhaust her own natural wealth as quickly as possible. This, Great Britain has done for a century or more—to her sorrow.

In this future, plywood is certain to come into its own on a tremendous scale. With moisture from the heavens, and energy from the sun, the forests will continue to grow long after domestic iron ore is a mere memory—unless man sets his mind on the task of exhausting the forests too—through economic mismanagement.

Visualizing this, it is easy to see a complex development in all directions—not only as to beams and girders but also as to every conceivable combination useful in home building and industry—and these are many. Some time ago a friend of the writer built a functional plywood home with double sheathing on 1-in. by 3-in. joists—all prefabricated and glued in his home basement during spare time, and then quickly erected. He was scorned for building a house of cards; but the proof of strength came quickly; a truck out of control ran into the house and pushed it sideways several inches without injury to the basic frame.

Experiences of similar order have multiplied recently; hence, papers on the theory and the use of plywood are highly welcome. Plywood will be of help to the builders of the future; and it will slow down the terrific rate at which iron ore is taken from the bowels of the earth.

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[2] "Technical Data on Plywood," Douglas Fir Plywood Assn., Tacoma, Wash., 1945.

[10] "Structural Application of Plywood," Douglas Fir Plywood Assn. Laboratory Repts., Tacoma, Wash., 1945.

[11] "ANC Handbook on the Design of Wood Aircraft Structures," U. S. Govt. Printing Office, Washington, D. C., 1943.

[12] Engr., Carbide & Carbon Chemicals Corp., South Charleston, W. Va.

[12a] Received July 30, 1946.

Professor Hansen has supplied one of the chapters most needed to make plywood history. He has written about I-beams; and he has written well. He has verified his formulas by experiments. Now the engineer can design plywood I-beams with greater confidence.

There is a weakness in such beams—as he states—which one may possibly eliminate. It is not the plywood, seemingly, that determines permissible unit shear stress, but the glue between web and flange; thus the plywood itself could do a great deal more work if its own strength were governing. Why not change the section? At present there is a small glue area as compared with a large flange area. It should be the other way around to get the most out of the section. Therefore, it is proposed that the beams—at least the heavier girder types—be built up as are steel plate girders, with fillet blocks (of plywood if preferable) to take the place of the four angles connecting flange and web plates. This would greatly increase the shear area holding the web to the flanges so that even with the relatively low shear strength of the glue much greater flange stress could be transmitted to the web.

Does Professor Hansen agree? Are not such girders made? If they are not made, why not? If they are made, what new formulas are needed to use them properly in structural plywood designs?

C. J. Hogue,[13] M. ASCE.[13a]—An interesting contribution to the data on a useful and efficient combination of lumber and plywood, utilizing the tensile and compressive strength of sawed lumber and the shearing strength of plywood is presented by Professor Hansen. As the author notes, this development is comparatively recent; and further information and test checks are needed. The paper offers methods of determining size and spacing of intermediate stiffeners, and of calculating deflection due to shear, which differ from those in "Technical Data on Plywood."[14] Through such presentation of different methods, the best will be found.

As information on wood is developed and wood is used more technically, its anisotropic character leads to complicated analyses. Formerly, a designer in wood need only be equipped with a formula for bending and an empirical, straight-line column formula. With present advances, there is need for simplification of design data.

A complicated analytical formula may often be expressed in simplified form within close accuracy, and charts and tables can be prepared for convenient determination of complicated factors. This trend tends to more convenient use of wood and plywood, to quicker calculation, and to safer design where an involved formula might be misinterpreted and solved erroneously. The writer is glad to note that this is done both in the paper and in "Technical Data on Plywood."[14]

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[13] Cons. Timber Engr., Seattle, Wash.

[13a] Received September 16, 1946.

[14] "Technical Data on Plywood," Section 9, Douglas Fir Plywood Assn., Tacoma, Wash.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

TOPOGRAPHIC SURVEYS

PROGRESS REPORT OF THE COMMITTEE OF THE SURVEYING AND MAPPING DIVISION ON TOPOGRAPHIC SURVEYS

Discussion

BY BENJAMIN E. BEAVIN

BENJAMIN E. BEAVIN,[4] Assoc. M. ASCE.[4a]—Obviously, this progress report is the result of a great deal of careful work on the part of the committee. In the first paragraph under the heading, "Function of a Topographic Map," a third and important use of topographic maps might be mentioned; namely, the use by the maintenance engineer whose problem, many times, is simplified by complete knowledge of conditions antecedent to construction. To the methods of developing topographic maps should be added a fifth one—(*e*) altimetry. Modern instruments may be used satisfactorily in combination with the other methods.

For highway work, a special plane-table board 24 in. by 36 in. is very convenient. Many highway surveys are plotted to a scale of 1 in. equals 50 ft. The 36-in. board will accommodate fifteen 100-ft stations with sufficient working room at the ends. This size of board also is useful in large-scale topographic mapping in developed areas; yet it is not particularly stable in high winds and is difficult to carry in rough country.

Horizontal and vertical control have been treated at great length in various manuals, but not always consistently. A progress report of this kind would be enhanced, if footnote page references to older specifications (for example, those for second-order traverse) could be included.

The writer would recommend that the following be considered for inclusion in the specifications given in Section A of the "Appendix:"

A-1. Datum.—In tidewater locations where no vertical control exists, a tide gage should be installed and observed for the longest practicable period,

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NOTE.—This report was published in April, 1946, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: October, 1946, by Roger E. Amidon, Daniel Kennedy, and Ralph P. Black.

[4] Associate, J. E. Greiner Co., Cons. Engrs., Baltimore, Md.

[4a] Received August 30, 1946.

in order to establish an approximate mean sea level. In other locations, a datum plane should be assumed.

A–2. Coordinate System.—Where it is desired to facilitate later application of the state grid to the resulting maps, the true meridian may be corrected to the state grid before computing coordinates.

A–3. Map Drafting.—Maps should also record the magnetic declination, average scale factor, latitude, and longitude.

A–4. Station Numbering.—As an alternative for numbers, names may be used for traverse stations, particularly where survey, inspection, and construction forces use the same data.

A–5. Field Survey Methods; Control System.—Several orders of traverse are included in control surveys. A doubled angle would not be appropriate for second-order work. Subtended angles make an acceptable check for chaining. With suitable instruments satisfactory control traverses may be run by this method.

A–6. Notebooks.—On large projects where time is at a premium, the work may be expedited by the use of loose-leaf notes. Computations and plotting may proceed while surveys are being completed, and the notes may be arranged in suitable order for filing (possibly by coordinates). To reduce the danger of loss, to increase the ratio of usable record space to heading space, and to permit filing in standard letter files along with computations, the writer uses 8.5-in. by 11-in. loose-leaf note sheets, printed on both sides. Computation books also may be loose leaf. Bench-mark and coordinate cards, 5-in. by 8-in., have proved very satisfactory. The writer files them by coordinates; but other systems are satisfactory.

A–7. Computations.—Computations should be checked by different persons and by a different method where practicable.

A–8. Monumentation.—A procedure that permits the early use of monuments is to set the base in concrete. In locations where it is advisable to set the top below the surface, the writer has used 4-in. or 6-in. scrap, cast-iron pipe, filled with concrete and with a bronze disk in the end. These monuments can be relocated with the aid of a dip needle or a mine detector.

A–10. Horizontal and Vertical Control.—

(*c*) What order of accuracy will be obtained by the specified method of measuring traverse angles?

(*g*) "Technical Procedure for City Surveys,"[5] specifies 0.017 $\sqrt{M}$ for first-order, and 0.04 $\sqrt{M}$ for second-order, leveling. Why is it necessary to introduce a new order midway between the two?

In closing, the writer would recommend strongly that survey methods for horizontal and vertical control, as outlined in previous manuals (particularly the specifications for the several orders of accuracy), be drawn upon freely for all new documents. If and when the older publications become obsolete, a revision obviously will be in order.

[5] "Technical Procedure for City Surveys," *Manual No. 10*, Manuals of Engineering Practice, ASCE, 1934.

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DISCUSSIONS

TORSION IN STEEL SPANDREL GIRDERS

Discussion

BY E. I. FIESENHEISER, EDWARD V. GANT, OSCAR HOFFMAN, AND PHIL M. FERGUSON

E. I. FIESENHEISER,[12] Assoc. M. ASCE.[12a]—The profession is indebted to Professor Lothers for presenting a method of analysis for torsional stresses in steel spandrel girders and particularly for Tables 1 and 2, which give torsion constants for use in design.

The analysis of steel frameworks for torsional stresses is a refinement not often used in steel building design, partly because shear stresses do not usually govern in the selection of floor beams. Ordinarily, attention is directed mainly to analysis for moment. Moreover, it is the writer's opinion that torsional analysis is not justified unless another factor of equal importance is also considered.

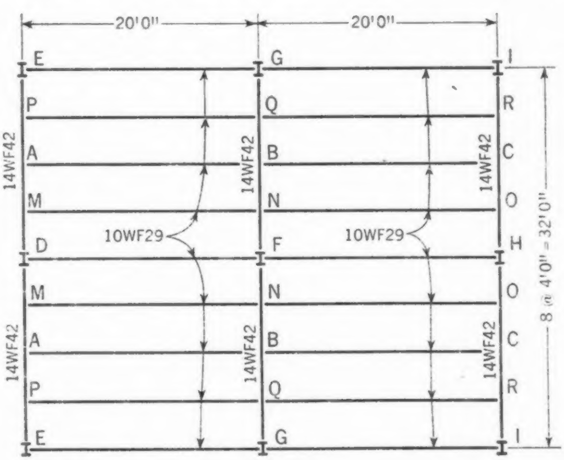


FIG. 9

The manner in which the ends of filler beams are connected to their supporting girders in actual practice is fully as important if greater refinement in design is to be used. Standard, riveted connection angles will probably be adequate to transmit the torque of a spandrel girder to its supporting column;

NOTE.—This paper by J. E. Lothers was published in March, 1946, *Proceedings*. Discussion of this paper has appeared in *Proceedings*, as follows: June, 1946, by John E. Goldberg, and I. Oesterblom, and October, 1946, by Robert V. Hauer.

[12] Associate Prof. of Civ. Eng., Illinois Inst. of Tech., Chicago, Ill.

[12a] Received July 13 1946.

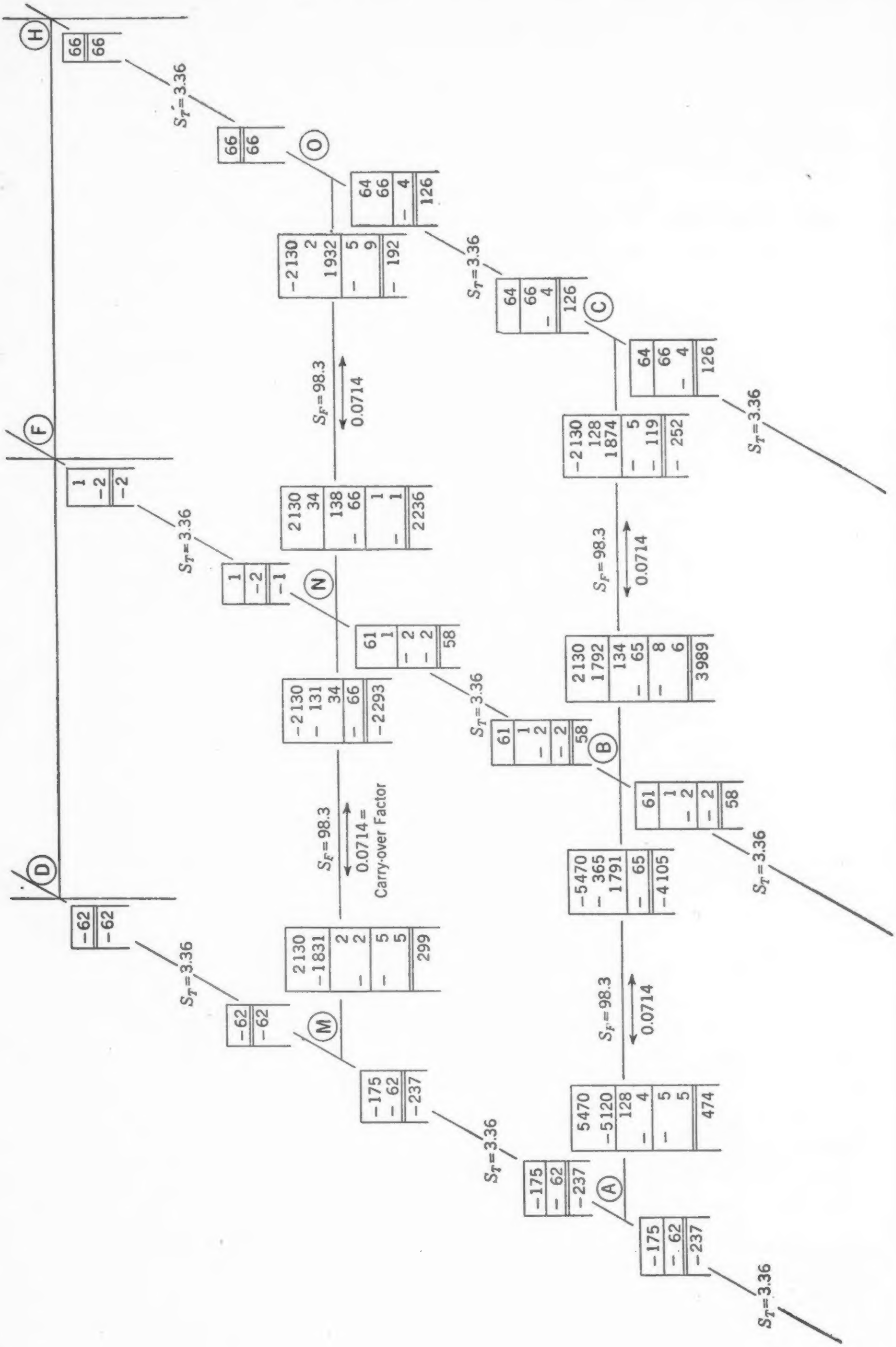


Fig. 10

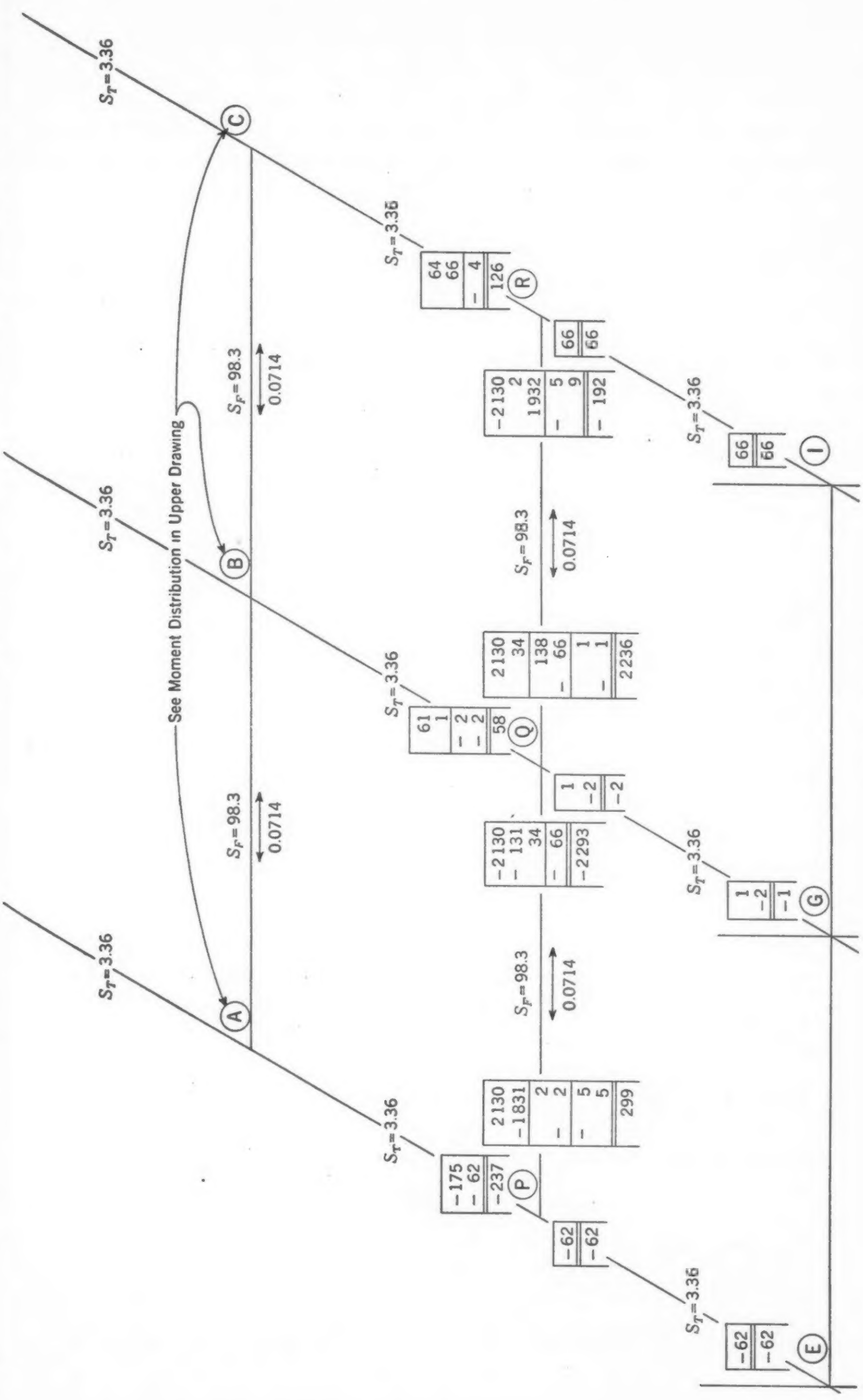


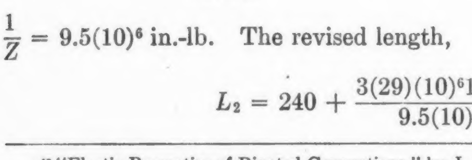
FIG. 10.—(*Continued*)

but, in beam and girder framing, the end connection angles will not be sufficiently rigid to transmit a large end moment into a joint without some reduction. End connections should not be assumed rigid unless special attention is paid to their design to make them so. Ordinary connections develop only partial continuity. Hence, it is common practice to design steel beams as simply supported, a practice justified only as being safe, and not as being economical. The writer proposes to show how actual connection restraint may be included in an analysis, if desired.

Referring to Example 3, Figs. 7 and 8, the author considers the connection of the 10WF29 filler beams to the girders to be rigid. The end moment of 50,998 ft-lb will then have to be resisted by the connection at point B. If an ordinary connection, consisting of four rivets in bearing on the web, is used, it will fail to resist this moment. The connection angles will deform or yield, requiring beam AB to resist a larger moment near the center of the span than it was designed to carry. Consequently, the beam will be overstressed.

As an example, the writer proposes to use the 10WF29 beams at 4-ft centers, as shown in Fig. 9. The distribution of moments in three cycles is demonstrated in Fig. 10, in which the order of distribution is: Joints A, B, C, M, P, N, Q, O, and R. Although an economical design for the beams does not result, neither does an overstress of the standard connection. This particular connection was tested by J. Charles Rathbun,[13] M. ASCE, who explained how connection restraint may be taken into account in an analysis. After a decade, this excellent paper is still very much to the point, and the methods given will be applied to the example, using moment distribution.

Because an end connection is not as stiff as the connected beam, the fixed-end moment, the carry-over factor, and the flexural stiffness of the member should be computed by taking into account actual conditions. The decrease in member stiffness enters the analysis by the use of a revised length of span, transformed by the connection. This revised length is termed L_2, and is equal to $L + 3\,E\,I\,Z$. The term $Z = \frac{\theta_M}{M}$ is the ratio of the angle of rotation θ_M of the end connection to the moment M producing the rotation. Its value is determined by test.



8-in. I-Beam @ 18.4 lb $2'\,11\frac{1}{2}"$ long

FIG. 11

A standard connection[14] for a 10-in. beam is to be used, as shown in Fig. 11. Assuming an average slope and with the assistance of available curves,[15] $\frac{1}{Z} = 9.5(10)^6$ in.-lb. The revised length,

$$L_2 = 240 + \frac{3(29)(10)^6 157.3}{9.5(10)^6} = 1{,}680 \text{ in.}$$

[13] "Elastic Properties of Riveted Connections," by J. Charles Rathbun, *Transactions*, ASCE, Vol. 101, 1936, p. 524.

[14] *Ibid.*, p. 527, Fig. 1, Specimen 2.

[15] *Ibid.*, p. 538, Fig. 13, Specimen 2.

The fixed-end moment for symmetrical loading is given by the expression:

$$(\text{FEM}) = \frac{6\,Q}{L}\,\frac{2\,L_2\,\frac{L}{2} - L\,\frac{L}{2}}{4\,(L_2)^2 - L^2} \qquad (15)$$

in which Q, the area under the moment curve for a simple beam, is equal to $\frac{w\,L^3}{12}$ for a uniform load of w per unit of length. Substitution yields a fixed-end moment of $\frac{w\,L^2}{60}$ for the 10WF beams. The carry-over factor $\frac{L}{2\,L_2} = \frac{240}{2 \times 1{,}680} = 0.0714$. The flexural stiffness—

$$S_F = \frac{3\,E\,I}{L_2 - \frac{L^2}{4\,L_2}} \qquad (16)$$

—yields $0.431\,\frac{E\,I}{L} = \frac{0.431 \times 29 \times 157.3}{20} = 98.3$.

In applying moment and torque distribution for space frame structures to to this example, the writer uses the process described by L. E. Grinter,[16] M. ASCE. The sign convention for moment or torque assumes that a moment or torque tending to produce clockwise rotation of a joint is positive.

The stiffness factor $S_T = \frac{G\,K}{L} = \frac{11.6 \times 1.16}{4.00} = 3.36$ for the 14WF 42 girders subjected to torque. The carry-over factor for torque is -1.0.

The revised load carried by the filler beams is now 320 lb per ft dead load, and 500 lb per ft live load. In the example to follow, beam AB only is loaded with live load. For this beam the fixed-end moments are $\frac{(320 + 500)(20)^2}{60} = 5{,}470$ ft-lb. For the other beams the fixed-end moments are $\frac{320\,(20)^2}{60} = 2{,}130$ ft-lb.

The numerical work was done by slide rule since any refinements in accuracy greater than 1% or 2% are of no significance. The distribution begins at joint A in Fig. 10 and proceeds through three complete cycles. The original fixed-end moment at joint A is 5,470 ft-lb. When this joint is released, it is permitted to rotate in a clockwise direction. The accompanying balancing moments are $-5{,}120$ for member AB, and -175 for members AM and AP, which are in proportion to the S_F-factor and S_T-factor for the members. The clockwise rotation at joint A induces a counterclockwise moment at end B of $0.0714 \times 5{,}120 = 365$, which is written in with the minus sign at joint B to indicate the direction. The balancing moment or torque of -175, accompanying the clockwise rotation at joint A, induces a clockwise moment at joints M and P, of 175, which is written in as $+175$ at these joints. A single straight line is drawn under each balancing moment to indicate completion of distribution. After all moment and torque have been balanced, a double line is drawn before adding the columns at the various joints.

[16] "Elastic Properties of Riveted Connections," by J. Charles Rathbun, *Transactions*, ASCE, Vol. 101, 1936, Vol. 103, 1938, p. 1512.

The maximum end moment to be carried by any connection is 4,121 ft-lb at end B of member AB, which is not too high for the ordinary connection to withstand.

The method of moment distribution applied to the analysis of space frames is more tedious, although scarcely more complicated, than is moment distribution applied to planar structures. Its use is justified in the investigation of torsional shearing stresses and in the study of connections.

Edward V. Gant,[17] Assoc. M. ASCE.[17a]—As the author has stated, one of the problems that arises in investigating the torsion of steel spandrel girders is the assumption regarding the rigidity of the connection between the floor beam and the spandrel girder. There is not much experimental information on this point available; but, according to recommendations made by the Steel Structures Research Committee of Great Britain,[18] no allowance should be made for a restraining moment at the end of a beam which frames into the web of a girder unless there is a similar beam framing into the girder on the opposite side at the same point. Thus, with ordinary steel construction there would seem to be little moment developed at the spandrel end of a floor beam, and consequently torsion in the spandrel girder from this source could be neglected. However, where welded connections are used, or where special provision is made for a restraining moment at the spandrel end of a floor beam, some moment may be developed; and an investigation of the resulting torsion in the spandrel girder would be desirable. In any case, a definite answer to the question of the rigidity of the connection between the spandrel girder and the floor beam would seem to await additional experimental evidence.

Assuming that the connection between the spandrel girder and the floor beam is rigid enough to transmit moment, Professor Lothers has proposed simple formulas (Eqs. 12) for the moment at the spandrel end of the floor beam. It should be emphasized that these formulas do not allow fully for the facts that the spandrel girder and the floor beam are units of a structural frame and that deformation of other members may influence their deformation. For instance, the rotation of the ends of the spandrel girder is a function of column deformation and could either relieve or increase the torsion in the spandrel girder, depending on the direction of rotation.

Another factor that may influence the validity of Eqs. 12 is the relative vertical deflection of the ends of the floor beam that frames into the spandrel girder. The effect of this deflection would be to add algebraically a moment to the fixed-end moment $(FEM)_{AB}$ in Eqs. 12. From Eq. 10 and a similar equation for the moment M_{BA} at the inner end of the floor beam, the corrections would be

$$\pm \frac{6\,E\,I\,\Delta}{L^2}; \qquad \pm \frac{2\,E\,I\,\Delta}{L^2}; \qquad \text{and} \qquad \pm \frac{3.3\,E\,I\,\Delta}{L^2} \quad \text{.......(17)}$$

for Eqs. 12*a*, 12*b*, and 12*c*, respectively. In many cases the magnitude of the correction will be small. However, the possibility of a large relative deflection modifying the moments in Eqs. 12 should not be overlooked.

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17 Asst. Prof., Civ. Eng., The Univ. of Connecticut, Storrs, Conn.

17a Received July 17, 1946.

18 "Final Report of the Steel Structures Research Committee," Dept. of Scientific and Industrial Research, His Majesty's Stationery Office, London, 1936, p. 552.

Oscar Hoffman,[19] M. ASCE.[19a]—A better understanding of the actual behavior of structural frameworks, beyond that which is given by simple two-dimensional analysis, is contributed by this excellent paper. The methods presented are very straightforward and workable in cases similar or closely related to those illustrated by the author; but they would become somewhat impracticable and cumbersome for a larger number of floor beams, say, three or more, within a single spandrel girder span.

The writer wishes to call attention to the possibility of using finite difference equations in handling such cases. The fundamentals of the calculus of finite differences, and some interesting applications of it to structural problems can be found in a book by Theodor von Kármán and M. A. Biot[20] published in 1940.

The method of approach to the spandrel girder problem can best be illustrated by its application to a comparatively simple case—a single-span spandrel girder with fixed-end supports (which may be visualized as an interior span of a continuous girder supported by infinitely rigid columns) carrying a number of floor beams, placed at constant intervals, a. The girder span is $l = N\,a$, in which N is an integer. The floor beams are so arranged that there is one of them at the center line of each supporting column. All floor beams are assumed to be fixed at their inner ends, to have the same moment of inertia, I, and to carry the same uniform load, w, per unit length. Fig. 12(a) shows a view of the spandrel girder. Figs. 12(b), 12(c), and 12(d) show the torques, M_x, acting on the girder at each floor beam intersection; the torsional moment, T_x, along the girder; and the angle of twist, θ_x. Fig. 12 shows a complete analogy with the load diagram, shear diagram, and moment diagram of an imaginary simple beam having the span length l and being loaded with the imaginary concentrated loads, M_x. Fig. 13(a) represents the forces acting on a typical floor beam and Fig. 13(b) is the corresponding moment diagram.

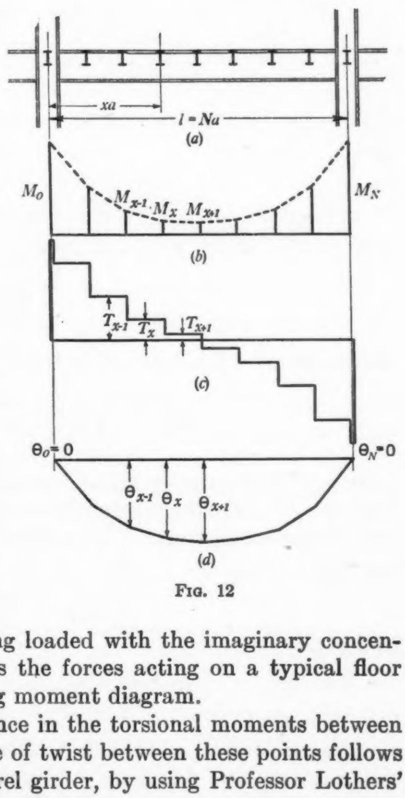

Fig. 12

The relationship between the difference in the torsional moments between x and $x - 1$ and the change in the angle of twist between these points follows from the elastic properties of the spandrel girder, by using Professor Lothers'

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[19] Associate Prof., Civ. Eng., Case School of Applied Science, Cleveland, Ohio.

[19a] Received August 2, 1946.

[20] "Mathematical Methods in Engineering," by Theodor von Kármán and M. A. Biot, McGraw-Hill Book Co., Inc., New York and London, 1940, p. 437.

notations, G and K:

$$\theta_x - \theta_{x-1} = T_x \frac{a}{G K} \quad \text{(18)}$$

At the same time, θ_x represents the outer end slope of the floor beam x, and M_x represents the outer end moment for the same floor beam. The relationship between those two quantities, readily obtained, for instance, from the slope-deflection equation, is:

$$\theta_x = \left(\frac{w L^2}{12} - M_x\right) \frac{L}{4 E I} \quad \text{(19)}$$

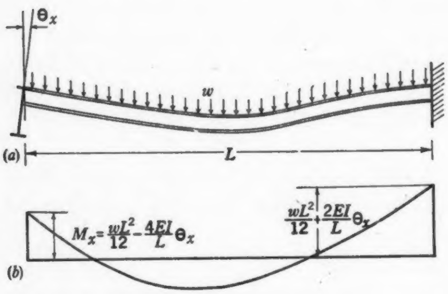

FIG. 13

A third basic relationship is that the change in torsional moment from a point to the left to another point to the right of the intersection of a floor beam with the spandrel girder is equal to the torque, M_x, developed by the rigid connection of the floor beam with the girder. Then:

$$M_x = T_x - T_{x+1} \quad \text{(20)}$$

Substituting in Eq. 18 the expression for θ_x given in Eq. 19 and a similarly built expression for θ_{x-1}, and solving for T_x, one obtains:

$$T_x = (\theta_x - \theta_{x-1}) \frac{G K}{a} \quad \text{(21)}$$

Substitution of this expression, and of a similar one for T_{x+1}, in Eq. 20 yields:

$$M_{x-1} - \left(\frac{4 a E I}{L G K} + 2\right) M_x + M_{x+1} = 0 \quad \text{(22)}$$

a linear difference equation of second order. The method of finding the solution of such an equation is discussed and abundantly illustrated by Messrs. von Kármán and Biot.[20] Its application to the present problem leads to the expression:

$$M_x = \frac{w L^2}{12} \frac{\beta^x + \beta^{N-x}}{\beta^N + 1} \quad \text{(23)}$$

in which

$$\beta = 1 + \frac{2 a E I}{L G K}\left(1 + \sqrt{1 + \frac{L G K}{a E I}}\right) \quad \text{(24)}$$

The assertion that Eq. 23 satisfies Eq. 22 and the boundary conditions of the problem ($\theta_0 = 0$ and $\theta_N = 0$) can be verified by simple substitution.

The expression for the torsional moment between the floor beams $x - 1$ and x is to be mentioned because of a subsequent use of it:

$$T_x = \frac{w L^2}{12} \frac{\beta^{N-x+1} - \beta^x}{(\beta^N + 1)(\beta - 1)} \quad \text{(25)}$$

Eq. 23 has some general implications. It shows that the behavior of spandrel girders depends on β, which is given by Eq. 24 as a function of the stiffness ratio $\frac{a\,E\,I}{L\,G\,K}$.

A survey of the possible range of practical limits of values for the stiffness ratio in steel structures, with the usual low torsional stiffness of the spandrel girder, shows that, in general, β can be approximated satisfactorily by the expression:

$$\beta \approx \frac{4\,a\,E\,I}{L\,G\,K} \qquad (26)$$

and, as the stiffness ratio is seldom less than 15, β will be larger than 60 in most cases.

In such cases, the end moment of the floor beam 1, M_1, and the end moment of the floor beam 0, M_0, will be related by the approximate formula:

$$M_1 \approx \frac{M_0}{\beta} \qquad (27a)$$

and, similarly,

$$M_2 \approx \frac{M_1}{\beta} \approx \frac{M_0}{\beta^2} \qquad (27b)$$

In other words, M_1 drops to less than 2% of M_0 and the end moments of the floor beams with order numbers from 2 to $N - 2$ are negligible, for all practical purposes, irrespective of the number of floor beams. Furthermore, the total torque transferred by the spandrel girder into the column can be estimated with an error of less than 2%, as being equal to M_0. In the example herein discussed, $M_0 = \frac{w\,L^2}{12}$ results from the assumption of infinitely stiff columns, but the foregoing considerations indicate the possibility of taking into account, with a fair approximation, a finite column stiffness in conjunction with a single floor beam —the one that frames directly into the column.

An essentially different situation arises with reinforced-concrete structures because of the higher torsional stiffness of the spandrel girders and of the consequently lower values of β. For such structures, β can vary from a little more than 1 to 5; but it seldom passes the latter limit. As a consequence, the moment, M_x, decreases slowly from its maximum value, at the columns; and its magnitude is no longer negligible in the design of the floor beams, even at the center of the spandrel girder span.

The total torque, transferred to the column, which forms an intermediate support for a continuous spandrel girder, is $M_0 + 2\,T_1$, and, in the specific example considered herein, the following expression can be found for it:

$$\frac{w\,L^2}{12}\,\frac{(\beta^N - 1)\,(\beta + 1)}{(\beta^N + 1)\,(\beta - 1)} = \frac{w\,L^2}{12}\,\nu \qquad (28)$$

In Eq. 28, the factor:

$$\nu = \frac{(\beta^N - 1)\,(\beta + 1)}{(\beta^N + 1)\,(\beta - 1)} \qquad (29)$$

is evidently the number of floor beams that develop the fixed-end moments to be transferred fully into the column. The product νa can be considered as the width of the tributary strip of floor which is to be taken into account in cases of finite column stiffness, with its moment of inertia equal to νI.

The writer wishes to emphasize the simplifying assumptions upon which the foregoing specific example is founded, and which are primarily responsible for the reasonably simple results obtained; yet, the writer feels strongly that the method of approach illustrated herein may have some broader applications, although the mathematics involved in such applications might prove to be somewhat more formidable.

PHIL M. FERGUSON,[21] M. ASCE.[21a]—The writer has watched the development of the torsion formula for steel beams and the attempts to apply it to practical problems with a great deal of interest. A solution of this problem is needed in many situations and the trend of architectural development each year adds to the need; yet the freestanding steel beam subject to torsion is relatively a rare condition. In building construction, there is nearly always a restraint present that is not considered in the conventional treatment of the problem, for example, a concrete floor slab or a stiff wall encasement.

The author's solution of the problem is interesting and helpful. However, it should be emphasized that its field of usefulness is limited to cases in which the girder is free to rotate in accordance with the simple torsion formula (Eq. 2). This eliminates most cases in which a concrete slab frames into the girder, including all cases where the girder is encased in concrete, and probably most cases where the slab is simply poured on or around the top flange of the girder.

In ordinary design the assistance that the slab gives to the beam may be, and generally is, ignored. The slab is likewise considered as an entirely separate unit for design purposes. However, a concrete slab resting on a steel beam increases the stiffness and reduces the deflection of the beam very materially, even when not accompanied by concrete fireproofing of the beam. It would thus seem necessary, in dealing with spandrel girder torsional stresses that depend chiefly on beam deflection, to consider the probable composite action of steel beam and concrete slab. The slab modifies the beam action in several ways:

1. The stiffness of the composite beam is much greater than that of the steel beam alone;
2. The increased stiffness greatly reduces the beam deflection and the end rotation that results, whether the spandrel girder is similarly reinforced or not; and
3. The neutral axis of the composite beam lies much higher—above the middepth of the steel beam.

The effect of these differences will be discussed after the action of the girder is considered.

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[21] Chairman, Dept. of Civ. Eng., Univ. of Texas, Austin, Tex.

[21a] Received August 29, 1946.

The slab also modifies the girder action. Even when the girder is not encased, and the web and bottom flange are thus left almost entirely free from concrete restraint, the top flange is usually interlocked with the concrete. Lateral movements of the girder top flange that can result are largely determined by the movements of the concrete; these movements depend on the slab deflection and the extent to which the slab is shortened by its action as part of the compression flange of the beam.

A comparison between the author's assumed girder flange deflection and the probable deflection when a concrete slab is present will reveal the difference. The simple torsion formula assumes that the upper girder flange is displaced laterally by the end rotation of the beam it carries, and that this displacement decreases to zero at the end of the girder at essentially a linear rate. In other words, a plan view of the deflected flange is assumed to be essentially a straight line between the beam connection and the end of the girder. The concrete slab, on the other hand, shortens lengthwise, with the beam axis and perpendicular to the girder, in an amount that is dependent upon the extent to which it serves as a compression flange for the beam. Generally this flange action will correspond to nearly perfect adhesion between the concrete and the steel beam. It is not probable that this concrete shortening will decrease directly with the distance from the beam, as would be required to permit the steel girder to satisfy the simple torsion formula. Instead, it might be expected that this shortening would be nearly uniform for a considerable distance on each side of the beam. The upper girder flange must be displaced to conform to this concrete pattern, as if the beam had a very wide connection tied in to the top girder flange. The result seems to be that the top girder flange is displaced almost uniformly over a considerable length, leaving a much shorter length within which it must return to its normal position (if it does so return at the end of the girder). At the same time, the bottom flange retains greater freedom of action. The resulting torsional resistance to this type of rotation and displacement must be considerably different from that indicated by the simple torsion formula.

It would also appear that the rotation impressed by the connection of the composite beam is a twisting about an axis at the level of the composite beam axis, and this is several inches above the middepth of the beam and the girder.

The rotation of the girder over most of its length thus seems to be about an axis which varies from point to point, and which is generally not through the centroid of the cross section. The torsional stiffness will surely be increased by this shifting of the center of rotation, since it adds a lateral displacement of the girder to the rotation about an axis through the centroid. This aspect of the problem seems to have been largely neglected in the engineering literature that has come to the writer's attention. In the related field of beams curved in plan, elaborate formulas have been developed which assume a beam to be free to twist about a centroidal axis; but no mention is made of the concrete slab which usually enforces entirely different deformations. This seems more serious than approximations for the value of K.

The effect of the concrete slab can be evaluated more definitely in its relation to the bending stiffness of the beam, which is greatly increased even when

the beam is not encased. Although the exact amount of increase may be subject to some uncertainties, a 4-in. concrete slab added to Example 2 would at least double the beam stiffness. This increase in beam stiffness would result in a reduction of approximately 50% in the total rotation of the girder envisioned in this paper; and girder web shears created by torsion would seem to be reduced proportionately. However, the writer is disturbed by the feeling that the complex type of rotation described herein would have a considerable effect on the resulting torsional stiffness and torsional stresses. For this reason, he does not state that torsional shears are necessarily less serious than those Professor Lothers would find; but they may be quite different.

If the spandrel girder is encased in concrete, its torsional action would be greatly modified, and an entirely different analysis would be necessary. Especially in this case, the slab moments on a section parallel and adjacent to the girder flange (caused by the deflection of the slab relative to the girder) would seem to be of considerable importance. When strictly considered, these slab moments complicate, very materially, the moment-distribution process indicated in Fig. 6.

If there is objection to the use of a composite beam as not the usual method of steel design, a different problem is presented, but one that is still without a simple solution. If the slab does not act as a flange of the beam, it would seem to follow that it would act as a relatively rigid spacer which would tend to hold the top flange of the girder in its initial position; and any girder rotation would be rotation about the top flange, not about the centroid of the girder.

In the form presented by Professor Lothers, the solution is thus subject to considerable question when concrete floors connect to the girder. Nevertheless, there are cases in which this analysis is directly applicable, such as open steel frameworks, timber flooring on steel beams, and the like. It is a step in the larger problem that must ultimately be solved, and the author is to be commended for presenting this step so clearly.

It seems to the writer that Eq. 12*b* is not mathematically correct, although the practical difference is not important. For Case II the effective stiffness of the beam (from joint A) is $\frac{3\,S_F}{4}$ and the end moment, before joint A is allowed to rotate, is $\frac{3}{2}\,(\text{FEM})_{AB}$. Then a moment distribution would give

$$M_{AB} = \frac{S_T}{S_T + \frac{3}{4} S_F} \times \frac{3}{2}\,(\text{FEM})_{AB} = \frac{2\,S_T\,(\text{FEM})_{AB}}{\frac{4}{3} S_T + S_F} \quad \ldots\ldots\ldots (30)$$

Eq. 30 differs from Eq. 12*b* only in the first term of the denominator, and this term is often so small that its complete omission would do slight damage.

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DISCUSSIONS

BALANCED DESIGN IN URBAN TRIANGULATION

Discussion

BY CHARLES D. HOPKINS

CHARLES D. HOPKINS,[12] ESQ.[12a]—In concluding this paper, the writer wishes first to thank the persons who have contributed to the published discussion. A large measure of thanks is due, also, to those individuals whose advice continues to be a helpful factor in the development of a better understanding of the subject. It seems certain that every opinion that is brought to bear during the unfolding of the concept will materially aid progress.

The most significant reactions to this exploratory effort have come from the engineers in other fields. Without any doubt, the tenor of their remarks indicates a desire to understand this increasingly important subject and a belief that the subject material must be expressed in relative terms in the common language of the engineer. There is the "rub," since the person endeavoring to assist the growth of fellowship is confronted with the ever present difficulties: First, to conceive the nature of the problem and an approach to solution; and, second, to express findings so that the details and the relationships are clearly understandable.

It seems clear, in the light of knowledge acquired since this paper was written, that the first need for better understanding will be met when the subject material is properly classified in the conventional engineering manner—(*a*) the properties of the materials, (*b*) the structure in theory, and (*c*) the structure in place. The materials in triangulation, whose properties have been under close investigation for many years, are found in the field books recording the observations. The structure in place involves the use of the various projections to define the position of adjusted stations on the earth's surface and the data for projections are available in great detail through the long continued work of the mathematicians; but, to fully realize the relationship between the materials for construction and the structures in place, it is absolutely necessary to develop

NOTE.—This paper by Charles D. Hopkins was published in November, 1945, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1946, by Walter S. Dix; and September, 1946, by Henry W. Hemple, L. G. Simmons, C. A. Whitten, and Ronald F. Scott.

[12] Geodetic Engr., Office, Chf. of Engrs., U. S. Army, Washington, D. C.

[12a] Received October 1, 1946.

the theoretical concept for the structure. Otherwise, the engineer can never expect success in final results with any degree of certainty.

The writer appreciates to the full the complexities in the subject taken as a whole. That circumstance needs no elaboration. However, it is possible to reduce any subject to a degree of simplicity, given a point of attack and the required time. In this connection, an example of the reduction of a minor subject in this field, possibly encouraging, can be examined as follows:

In the quadrilateral with both diagonals, the geometric conditions are found stated in four angle equations and five side equations. These can be put into the adjustment as three angle equations and one side equation, or two angle equations and two side equations, or one angle equation and three side equations. The number of combinations of the four equations required in this simplest type of completed unit figure is somewhat surprising—a total of 126 combinations has been identified, of which 120 are true and useful and six are false and useless; but, of the large number of useful combinations, only one combination is required, usually three angle equations and one side equation being the selected choice.

This comparison suggests that an understanding of the minimum working essentials can be grasped with relative ease. In other words, the engineer who builds roads does not necessarily need to remember the knowledge required to make the machines used in the operation.

In the light of experience, it is the writer's personal conviction (no doubt shared by many others) that unsuspected facts, indispensable to success, are present in triangulation. It does not seem reasonably possible to consider triangulation other than as a structure; since it can be defined as a structure, an arrangement of parts is involved—parts being the lines between stations. Therefore, since an arrangement of parts is an operation in fact, an orderly reasoned arrangement of these parts is logically necessary. This most strongly suggests an engineering treatment; and, the subject being treated as engineering, it becomes essential that every required principle in engineering be incorporated in the structure.

In view of the natural difficulties and the conflicting opinions relating to the geodetic field, it seems fitting to close this paper with an appropriate statement which will sum up the problem and point the way to an understandable solution. In one of the truly profound sayings of modern times, Sir Norman Angell poses the question: "How, out of complexity, to distill simplicity; out of knowledge, essential understanding; out of confusion, clarity." It seems entirely reasonable to state that the last mentioned attributes—simplicity, essential understanding, and clarity—applied to their limits would go far toward making the geodetic subjects more understandable, and certainly more attractive, to the engineering profession.

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DISCUSSIONS

DRAWDOWN TEST TO DETERMINE EFFECTIVE RADIUS OF ARTESIAN WELL

Discussion

BY N. S. BOULTON

N. S. BOULTON,[11] ESQ.[11a]—The importance of carefully recording both the small variations in pumping level, which may occur during pumping tests at constant discharge, and the duration of the test, are appropriately stressed in this paper. From such information it is possible to predict, as the author has shown, the probable steady decline in specific capacity "for periods of several months or a few years" when the well is pumped at constant discharge. It is important to remember, however, that the accuracy of this prediction depends essentially on the assumption that the compressibility of the aquifer (which enters into the coefficient of storage) has the same value for the very small pressure releases which occur at large distances from the pumped well as for the comparatively large pressure releases near to the well. It would be appreciated if the author could present evidence in support of this assumption, based on long-period observations of declining well levels. In addition, it would be interesting to know whether the author has been able to check the values for "well loss" by direct estimates of the pipe friction loss as the water flows inside the well casing and also of the loss of head due to the screen.

For the fourth period of the test at Bethpage, Long Island, N. Y., the depth of water in the well was apparently about 238 ft. Allowing for the water entering the well uniformly along the bottom 50 ft, a reasonable estimate (from a usual formula) for the head lost in pipe friction in the 8-in.-diameter tube is about 10.5 ft, including 1.5 ft for the velocity head. The computed well loss (see heading, "Data from Multiple-Step Drawdown Test") is stated to be 15.5 ft, which leaves 5 ft for the loss due to the screen. It is easy to calculate the latter loss on the assumption of flow through a uniform permeable medium outside the screen to which Darcy's law may be applied. Thus, for long vertical slots spaced equally around the circumference of the well, it can be shown from the potential solution for the flow net that the head loss due to the restricted

NOTE.—This paper by C. E. Jacob was published in May, 1946, *Proceedings*.

[11] Lecturer in Chg., Dept. of Civ. Eng., Univ. of Sheffield, Sheffield, England.

[11a] Received September 25, 1946.

inlet area provided by a slotted tube is closely given by:

$$h = \frac{Q}{2\,\pi\,N\,b\,k}\log_e\left(\frac{2}{1-\cos\nu\,\pi}\right)\quad\quad(26)$$

in which N is the number of vertical slots around the circumference of the tube; and ν is the slot-width ratio or width of slot divided by the distance between the centers of two adjacent slots.

According to Eq. 26, the head loss is proportional to the discharge and, for a given slot-width ratio, inversely proportional to the number of slots.

If $Q = 3.39$ cu ft per sec and $b = 50$ ft, as in the fourth period of the Bethpage test, and if $k = 0.004$ ft per sec (as deduced from Fig. 8), assuming $\nu = \frac{1}{4}$ (since the dimensions of the slotted tubing are not given in the paper), it is found on substitution in Eq. 26 that $h = 5.2/N$ ft. For one hundred slots, each 0.063 in. wide, uniformly spaced around the circumference, $h = 0.052$ ft which is negligible. On the other hand, if the slots are arranged in batteries numbering, say, ten in the circumference, the batteries being 0.5 in. wide with 2 in. between them, $h = 0.52$ ft which is still small.

It should be emphasized that this calculation makes no allowance for any clogging of the slots. Such clogging may account for the discrepancy between the small calculated screen loss and the value of 5 ft deduced from the test result.

Corrections for *Transactions:* In May, 1946, *Proceedings*, on page 629, change the title to read "Drawdown Test to Determine Effective Radius of Artesian Well"; on page 636, line 26, delete ", given in Fig. 4," in the formula for β', lines 31 and 32, change "sq ft" to "cu ft" and "sq in." to "cu in."; on page 639, line 39, change "$t = 0$" to "$t = t_0$"; on page 642, line 10, change "7(a)" to "7(b)"; in Figs. 2 and 3 change "μ" to "u" and "$r^2\,s$" to "$r^2\,S$"; in the left-hand ordinate caption of Fig. 5, delete the exponent (^{2}); in Fig. 6 change Roman numeral superscripts to primes of the same order; and correct the last column heading in Fig. 8 to read "$\Delta Q_{i-1} + \Delta Q_i$." On page 639, change lines 30 to 38 to read "The light lines in Fig. 6(b) show the drawdown that would occur during the successive periods of the test if there were no well loss; and the heavy lines, to which the notations refer, include the well losses, $C\,Q^2$, values of which are indicated in Fig. 6(a)."

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DISCUSSIONS

RIGID-FRAME STRUCTURES SUBJECT TO NONUNIFORM THERMAL ACTION

Discussion

BY I. OESTERBLOM, AND FRANK R. HIGLEY

I. OESTERBLOM,[4] M. ASCE.[4a]—It is a sign of wakening to important but neglected facts that thermal action in structures has been more frequently discussed recently. There is again a sample of this trend in the publication of the paper by Mr. Tommerup. The author has succeeded in presenting a method that is quite applicable to simple frames. This commentator agrees wholeheartedly and thankfully with the logic of the method, but believes that the details must be modified to make the method truly useful.

There seem to be two basic points that require more thought and possibly correction—and some minor points, also. One of the basic points—if the author should agree—would materially lower the stresses; the effect of the other is the reverse. Presumably, the author has neglected the second point with the very good intention of simplifying his method; but, in doing it, he may lead novices astray unless he issues at least a warning. One may well question if the omission can be made fairly, when it is connected so intimately with the designer's problem that a separate superimposed solution for the omission is not permissible.

The two items may be presented, as follows:

(1) When heat is added on one side there is not only elongation and compression on that side, but also tension on the opposite side; and

(2) The direct longitudinal forces cannot be ignored, for they are very large and also set up bending moments by acting over a curved and deflected element.

Contrary to the conclusion of Mr. Tommerup, a correction for tension on the opposite side (item (1)) yields a deflection. Item (1) is predicated on the necessity of having a tensile reaction to the compressive stresses on any small section dx exactly in that element, and that condition sets up a neutral axis in the center line. Otherwise, where could the reactions be? They must be in-

NOTE.—This paper by Carl C. H. Tommerup was published in June, *Proceedings*.

[4] Engr., Carbide & Carbon Chemicals Corp., South Charleston, W. Va.

[4a] Received July 25, 1946.

cluded somewhere, and analysis of the element dx would indicate that they must be there, regardless of the fact that the accumulation appears as a moment at the corner. This also sets up a longitudinal shear, which the author—quite correctly from his point of view—has omitted.

Figs. 2, 3, and 4 are correspondingly subject to criticism, and need correction. Fig. 2 should show center lines of the elements; the deflection should apply to these lines. The gap and the deflection in Fig. 3 should be similarly adjusted, and so should the displacements in Fig. 4. It follows then that the stresses would be different—roughly, about one half of the values shown.

There is also doubt as to the value of E assumed for the rationalized concrete that is made today. With the aggregates properly selected, graded, and proportioned, and with the quantity of water under control, E is more likely to be nearer to 3.5×10^6 than the 2×10^6 as assumed[5]—thus a reduction of the concrete stress approximately to $\left(\frac{1}{2} \times \frac{2}{3.5} =\right)$ 29% of what is shown. The revised value of 600 lb does not seem so devastating as the 2,070 lb computed by Mr. Tommerup; yet, the revised value must be increased considerably due to the direct normal strains and stresses.

To compute these corrections the designer must know the initial construction temperature in addition to the 100° assumed at the time of the thermal addition of 400°. If the frame were built in cold weather with the temperature at 40° a rise of (60° + 200° =) 260° would be required to reach the average of the frame at that time. With the corners fixed against translation the stress is $\delta = E\,\Delta t\,\alpha = 3.5 \times 260 \times 7.9 = 7{,}200$ lb per sq in., which is to be added to the previous concrete figures, algebraically. As a result there is compression on the heated side equal to (7,200 + 600 =) 7,800 lb and compression, also, on the cool side equal to (7,200 − 600 =) 6,600 lb. In addition, there are the bending stresses caused by the thrust being applied to an independently created eccentricity.

To compute this bending is a very laborious task. Almost the only way to combine the two problems is by the aid of Castigliano's second law, which presents such difficulties as to make the simple method proposed by Mr. Tommerup very attractive. For this reason the writer sincerely hopes that his argument concerning the deflection can be proved erroneous.

There is a saving feature which bears directly and heavily on the high stresses—in actual construction it is difficult to establish complete fixation against translation. Thus, if the elements in question could be completely freed from external restraint there would be no direct thermal stress; and, depending on how much restraint there is, the stresses would vary between 0 lb and 7,200 lb.

This brings to the fore a point that Mr. Tommerup has demonstrated so well and so fairly: Investigators concerned with thermal forces must know how they act, and then must design the details so that these forces are nil, if possible, or at least very small. It is only by proper details that a structure can be

[5] "Johnson's Materials of Construction," by J. B. Johnson, rewritten and revised by M. O. Withey and J. Ashton, John Wiley & Sons, Inc., New York, N. Y., 8th Ed., 1939, p. 479.

prevented from cracking or crumbling or even complete disaster. This is an extremely important point to which very few designers give proper emphasis.

Gratitude is due Mr. Tommerup; structural engineers are fortunate that he has presented this phase of a complex subject so well. He has advanced it in the right direction; but he has still left much work to be done to amplify his method. As presented, it does not seem to be complete enough for safety. When it is finished no one should be able to charge a thermal failure in a simple frame to "the act of God" or "force majeure," as is so often done today by the formula fraternity inside the profession.

Frank R. Higley,[6] Esq.[6a]—A sheet metal unit developed in 1944 for a mechanical cycling operation responsive to nonuniform thermal action serves to demonstrate the possible danger from thermal stresses in unequally heated structures. The unit (see Fig. 18) comprises a single piece of steel, 0.040 in. thick and 5 in. long which, secured at one end, deflects its opposite end laterally 0.20 in. in 60 sec against a 1-lb load responsive to a minimum size gas pilot flame, substantially independent of ambient temperatures up to 1,000° (these data being approximate).

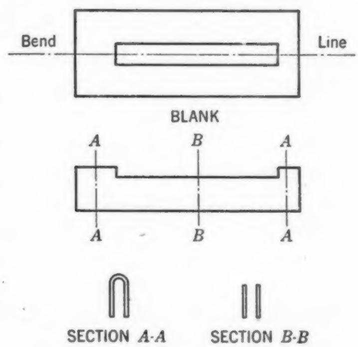

Fig. 18.—Thermally Responsive Unit.

With one objective in mind, it is often enlightening to study efforts leading toward the opposite objective. Of course, the purpose of designing such a thermal unit was opposite to that of designing the structures so ably analyzed in the paper. The sheet metal thermal unit is provided with a single slot and is simply bent along the line of the slot to provide a pair of members disposed in a parallel, slightly spaced, relation, with an integral interconnection adjacent to their ends along one edge of the resultant unit.

The principal features of the unit are: (*a*) Heat transfer between the members is inhibited to the greatest extent possible so that the thermal differential may be as great as possible; (*b*) the members have positive rigid interconnection at their ends to make best use of the relative expansive forces set up in the heated leg; and (*c*) while remaining rigid in the direction of generated force, the members are arranged to permit flexure in the resultant force direction desired.

Application of the same considerations to the type of structure described by the author indicates that (1) the wall should first be made laminate in so far as possible, to provide stratification between high-temperature and low-temperature faces; and (2) adjacent extremities of the laminas should be free from each other in the direction of their relative expansion. The author has at least implied the former of these two considerations under the heading, "2.—Rectangular Frame"; and has, in effect, quite thoroughly analyzed the second.

[6] Cleveland, Ohio.
[6a] Received September 13, 1946.

Inherently, from both the paper and the foregoing discussion, it appears that a plane of lamination, which in section determines what may be considered as a neutral thermal axis, should as nearly as possible be located to coincide with the common neutral axis of gyration between the same members at that section. Thus, as far as possible the members may have free relative expansion under the superimposed temperature differential, with relative expansion only on one side and relative contraction only on the other side of the axis.

Correction for *Transactions:* In June, 1946, *Proceedings*, page 761, line 17, change "80′" to "80 in."; on page 769, change line 17 to read "elastic curve is illustrated in Fig. 11(*d*) as compiled from Fig. 11(*c*); and add Fig. 11(*d*) to Fig. 11."

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DISCUSSIONS

MODEL STUDY OF BROWN CANYON DEBRIS BARRIER

Discussion

BY CLIFFORD A. BETTS, AND ROGER E. AMIDON

CLIFFORD A. BETTS,[4] M. ASCE.[4a]—The debris bed-load investigations constitute an interesting phase of the model studies of the Brown Canyon Barrier. They furnish information that may be helpful in the design of other structures or series of barriers for channel control. Particularly significant is the 1.73% to 2% slope of the debris upstream from the barrier. It indicates an ultimate valley storage of three quarters of a million cubic yards of debris upstream from the structure and above spillway level, or nearly double the crest level storage capacity. This occurs in a foothills arroyo having an average stream bed slope of 2.7%. The fact that the flood (4,400 cu ft per sec) of January, 1943, subsequent to construction, corroborated the model study predictions, adds to their value.

The use of channel barrier dams in narrow canyons, to stabilize mountain sides by holding the channel regimen and thereby preventing channel undercutting, is not new. However, there is need for more authoritative data on the complicated problem of predicting the behavior of bed loads of various materials under different volumes of flow.

In this case the bed load consisted of Lowe grandiorite, San Gabriel formations, and Wilson diorite, graded from fines to boulders, 3 ft or 4 ft in diameter, with sizes less than 6 in. predominating. As was to be expected, deposition in one bar might be localized fines or all coarse gravel; but the over-all distribution from the upper to the lower end of the canyon was relatively uniform, with the proportion of large-size material decreasing downstream. Differences between the behavior of the model and the prototype resulted from the artificial fixed channel, lack of tree-root obstructions, and the fact that no attempt was made to introduce material smaller than would be retained on a 200-mesh sieve ($\frac{1}{8}$ in. and smaller in the prototype) because of suspension. Nevertheless, there has been sufficient evidence in the subsequent behavior of the prototype channel to confirm the findings that the slope of the stream bed

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becomes greater (1) when debris-carrying flows are reduced, (2) when debris quantities are increased, and (3) when the size of bed-load debris is increased.

The introduction of the bed load to simulate actual watershed conditions seems to be an important factor. It requires meticulous study and calibration in order that the quantities applied to the various flows approximate the quantities of material actually transported.

Since these quantities increased with the rate of flow, it is obvious that extensive tests covering wide ranges of hydrographs would have added greatly to the information in this field had the project requirements justified the extra cost. As usual, however, this load will have to fall on some larger project.

Roger E. Amidon,[5] Assoc. M. ASCE.[5a]—The interesting model study of the Brown Canyon Debris Barrier is well presented in the paper under discussion. Valuable information pertaining to the spillway and downstream scour action was obtained. However, it is regrettable that more definite data did not result relating to the movement of debris and to the probable ultimate debris slope to be stabilized by the structure.

The original estimate that debris would assume an ultimate slope above the structure approximately equal to 70% of the original channel slope was based on measurements made above existing structures. These measurements were made during the period 1939 through 1941 at sites of approximately one hundred debris dams and check dams with small drainage areas above, seldom exceeding 5 sq miles. The age of the structures varied from 1 to 15 years. In most cases the channel gradient was affected from the site of the dam to the toe of the next upstream barrier. In some cases deposition at the toe of the next upstream structure was several feet deep. The dams included in the study vary from 2 ft to 35 ft in height, and are spaced from 20 ft to 2,500 ft apart. The original channel slopes varied from 4% to 35% and the debris slopes varied from 0% to 21%. The debris slopes varied from 0% to 100% of the original channel slopes, and averaged 52% for all barriers surveyed. The average debris slope above sixty-seven dams, where the original channel slope was 14% or less, was 66% of the original channel slope. On this basis it seemed reasonable that over a much longer period of time, on the lower gradient channels, the debris slope would at least approach 70% of the original.

The great number of variables that affect the slope which debris will assume above barriers forestalls the possibility of making precise estimates based on generalized curves for any specific structure. The size and quantity of material available for transportation, slope of the channel, volume of flow, obstructions in the channel, and period of years are some of the influencing factors.

In planning debris barriers it is extremely important to estimate correctly the volume of slope storage that will result in order to determine the economic feasibility of the structure. Detailed studies of field conditions and accurate measurements of progressive barrier storage, over a period of time, will provide data from which reasonably accurate estimates of debris slope can be made.

The increasing importance of the control of erosion and debris movement above reservoirs designed for flood control, water conservation, and hydroelectric power generation justifies serious consideration of this problem.

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